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THE BENEFICIAL USE OF STORM WATER

C. W. Mallory

Hittman Associates, Incorporated
Columbia, Maryland

January 1973

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Environmental Protection Technology Series

The Beneficial Use of Storm Water



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Mallory, C. W.

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A system study was conducted to determine the technical and economic feasibility of using small storage reservoirs throughout an urban community as a means of storm water pollution control. Facilities were provided to treat the water prior to release or to provide sub-potable or potable water for use in the community. A conventional approach to controlling storm water pollution was defined for comparative purposes.

Computerized system analysis was used to select the optimal combinations of reservoir locations, type of treatment, and type of reuse on a least cost per day basis. Alternatives were ranked and the optimal practical solution determined considering the constraints. It was determined that the use of local storage and treatment does represent a feasible and economical method for storm water pollution control. Further, the use of the treated water can supply a large portion of the fresh water demands of a typical urban residential community.

A demonstration program was planned and subsequently implemented to evaluate erosion and sediment control practices which includes a three-and-one-half-acre lake, evaluation of cleaning and sediment handling methods, and sampling and gaging stations to monitor changes in water quality and hydrology during urban development. (EPA)

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Charles W. Mallory

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THE BENEFICIAL USE
OF STORM WATER

By

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Contract No. 68-01-0173
Project 11030 DNK

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ABSTRACT

This report covers work originally performed by Hittman Associates in 1967 and 1968 under Contract No. 14-12-20 for the then Federal Water Pollution Control Administration. Only a limited number of copies of the report covering this work were produced at that time. The purpose of this report is to make this information available for general distribution.

A system study was conducted to determine the technical and economic feasibility of using small storage reservoirs throughout an urban community as a means of storm water pollution control. Facilities were provided to treat the water prior to release or to provide sub-potable or potable water for use in the community. A conventional approach to controlling storm water pollution was defined for comparative purposes.

The study considered an 1140-acre watershed located in the new city of Columbia, Maryland. Regression analysis techniques were used to develop hydrologic models for predicting storm runoff following development. Special water quality classifications were defined for the use of treated storm water and estimates were made of the water demands as a function of quality. Design and cost data were developed parametrically for storage, treatment facilities, distribution systems, and operation and maintenance as inputs to the system analysis model.

Computerized system analysis was used to select the optimal combinations of reservoir locations, type of treatment, and type of reuse on a least cost per day basis. Alternatives were ranked and the optimal practical solution determined considering the constraints on land use imposed by existing development plans.

As a result of this work, it was determined that the use of local storage and treatment does represent a feasible and economical method for storm water pollution control. Further, the use of the treated water can supply a large portion of the fresh water demands of a typical urban residential community.

A demonstration program was planned and subsequently implemented by the State of Maryland and the Environmental Protection Agency to evaluate erosion and sediment control practices on a 200-acre watershed in Columbia, Maryland. The project includes a three-and-one-half-acre lake, evaluation of cleaning and sediment handling methods, and sampling and gaging stations to monitor changes in water quality and hydrology during development.

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SECTION I

CONCLUSIONS

Based on this system study, it was determined that the use of small storage basins dispersed throughout an urban community for the control of storm water pollution was technically feasible and economically attractive compared to other alternatives. Further, the storm water collected in these basins could be treated to provide approximately half the water demands of a typical urban residential community. In addition, a number of secondary benefits could be derived through the use of this approach.

Four types of systems for the control of storm water pollution and for supplying water were defined for evaluation purposes. These were:

1. Use of local storage basins with treatment to body contact recreation standards for release downstream.
2. Use of local storage basins with treatment to sub-potable quality for reuse with separate distribution systems.
3. Use of local storage basins with treatment to potable quality for distribution through existing water distribution systems.
4. Use of a large basin to collect storm water from interceptors and pumping stations with conventional treatment systems.

The comparative costs and performance characteristics of the four systems are summarized as follows:

	(1)	(2)	(3)	(4)
Initial Costs (\$)	\$830,000	2,598,000*	1,445,000	1,315,000
Fixed Costs (\$/day)	119	373	207	189
Operation and Maintenance (\$/day)	55	151	194	68
Daily Costs (\$/day)	174	524	401	257
Amount of Water Supplied (gal/day)	0	556,000	460,000	0
Value of Water (\$/day)	0	207	250	0
Net Operating Costs (\$/day)	174	317	151	257
Cost per Year (\$/yr)	63,400	115,800	55,200	93,000
Cost per Acre/Year (\$/acre/yr)	55.25	104.30	48.05	81.71
Cost per Dwelling Unit (\$/D.U.)	16.88	30.82	14.68	24.97

*Includes water distribution system at \$1,562,000

The other major conclusions from this study are as follows:

1. A system of 10 small reservoirs dispersed throughout an 1140-acre watershed was found to provide storage capacity to receive in excess of 90 percent of the runoff of storms with a one year return interval without encroachment upon the existing land development plan.
2. The same effectiveness could be obtained with a lesser number of larger reservoirs at a reduced cost, if areas for the collection and storage of storm water were provided in the initial land development planning.
3. The amount of storm water that can be collected and treated could provide up to 52.5 percent of the water demands of a typical urban residential development. The most economical method of distributing the treated storm water would be to treat the water to potable quality and inject the water into existing water distribution systems.
4. Water treated to sub-potable water quality could provide up to 46.5 percent of the water demands of a typical urban residential area; however, the cost of providing a second distribution system for the sub-potable water would be much greater than the cost of treating to potable water quality for distribution through existing systems.
5. Treatment to sub-potable quality for reuse could be economically attractive in those cases where a large demand for sub-potable water exists for a few industrial or commercial establishments, and the cost of separate distribution systems can be reduced.
6. The use of small earthen dams for the construction of storm retention ponds is by far the least-cost alternative. The cost of constructed storage facilities of other types is generally prohibitively expensive.
7. The small reservoirs used to collect storm water will also be effective in controlling the excess runoff resulting from urbanization. With proper design, the runoff hydrographs can be maintained near those of a natural area.
8. The ideal time to introduce storm retention ponds is prior to development of the area and to use the ponds to control the erosion and sediment generated by construction.
9. Using retention ponds for storm water management and for sediment and pollution control will materially assist in the preservation of ecology in urban areas.

10. The use of collected storm water in sewerred communities could reduce nutrient releases since the water could be effectively diverted to treatment facilities having nutrient removal capabilities.
11. The use of storm retention ponds could be used for the separation of combined sewers in selected situations.

SECTION II

RECOMMENDATIONS

It is recommended that storm retention ponds be considered in the planning of the storm drainage facilities for new community development and in modifications of the storm drainage systems in existing communities. The ponds will provide an effective means of removing sediment and dampen the peak storm water flows resulting from urbanization. The availability of storage will reduce the capacity and costs of other drainage and treatment systems.

It is recommended that storm retention ponds be introduced prior to initial grading of the development site. This will provide a means of trapping the large quantities of sediment generated by construction. Following construction, the ponds should be cleaned and used as part of the permanent system for storm water management and pollution control.

It is recommended that demonstration projects be conducted to provide information on the design, application, operation, and maintenance of storm retention structures. Emphasis should be placed on methods of cleaning and maintaining the ponds since the acceptability of this approach will depend upon maintaining the ponds in an aesthetically acceptable condition at reasonable costs. These projects should also consider methods for the handling, processing, and disposal of sediment removed from the ponds.

It is recommended that demonstration projects be conducted to develop design and operating criteria for the application of storm retention ponds as a method of storm water management. Design and performance information should be obtained on relationships among pond levels, surface area, intensity of storm events, and the quantity and quality of water released. This should also include demonstration of retention structures operating in a series-parallel arrangement to define interactions and design and performance criteria for systems incorporating a number of retention structures.

It is recommended that a demonstration project be undertaken to determine the feasibility of treating storm water for various reuse applications. In this project, the emphasis should be placed on the development and demonstration of a water treatment system capable of reliably treating water to a quality suitable for distribution in existing water supply distribution systems. The treatment system should be automated for unattended operation and should have features for monitoring and automatic shutdown if water quality is not suitable for supply.

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SECTION III

INTRODUCTION

GENERAL

Simply stated, the local storage, treatment, and reuse of storm water is a concept for the control of storm water pollution in which the storm water runoff is collected in small storage basins dispersed throughout an urban area, treated to remove pollutants, and further treated for use. With this concept, the benefits derived from the use of storm water are used to offset the cost of effecting pollution control of storm water. This final report on "The Beneficial Use of Storm Water" contains a description of the system study, design, and evaluation of the local storage, treatment, and reuse of storm water and the analysis of the economic and technical feasibility of this concept.

The system study is based on the new city of Columbia, Maryland, and specifically Wilde Lake and its associated watershed which is located in the first village of Columbia. Wilde Lake is an artificial lake constructed in 1966, surrounded by apartments, townhouses, and individual residences, now approaching completion. This study is further based on the land development plans for this area, established lot boundaries, and other physical, aesthetic, and sociologic requirements existing within the area. The objective of this study has been to develop systems that could control the storm water pollution of Wilde Lake and to perform a comparative evaluation of the performance and economic aspects of these systems.

USE OF SYSTEM ANALYSIS

In order to subject the local storage, treatment, and reuse of storm water to comprehensive examination, system analysis techniques were used. This required the development of models to describe the various subsystems and compilation of generalized input data to define the design, economic, and performance parameters involved. The subsystems that comprise the overall system and the interrelationship to the characteristics of the study area and other utility systems are shown diagrammatically on Figure 1. Rainfall and water from a public water supply are inputs to the system and evapotranspiration, soil percolation, surface drainage out of the watershed, consumptive uses, and sanitary sewer flow are outputs. The principal parts of the overall system considered in the analysis are:

Storm Water Storage

Storm Water Pretreatment

Final Treatment

Treated Water Storage

Treated Water Uses

Following development, the system models were then applied using the data and characteristics of the Wilde Lake watershed in Columbia, Maryland. The formulation of the models and the results from applying these models to a specific watershed are contained in this report. It is noted that the methodology and the models used in this study are generalized and can be adapted to a variety of situations.

The objective function used in the formulation of the system model is described as follows:

Water released to surface drainage from the Local Storage, Treatment, and Reuse of Storm Water System shall meet or exceed stated effluent standards, and;

This condition will be achieved at the lowest net system cost.

This function is subject to all applicable physical, technological, legal, and institutional constraints, as described in the following sections. The term "lowest net system cost" is understood to be interchangeable with the term "highest net system benefit" and is defined as:

$$\min(\sum C_a + a \sum C_f - \sum B_a) \quad (1)$$

or, stated differently

$$\max(\sum B_a - \sum C_a - a \sum C_f) \quad (2)$$

where:

- C_a = marginal annual operating and maintenance costs
- C_f = marginal fixed construction and project costs
- B_a = marginal annual benefits
- a = capital recovery factor

Equation 2 is described as the "net benefit function" and subsequent discussion will be in terms of maximizing this function. The capital recovery factor used in Equation 2 is defined as:

$$a = \frac{i}{1 - \frac{1}{(1+i)^m}} \quad (3)$$

where:

i = annual rate of interest or cost of capital

m = period of amortization

SITE SELECTION

Early in the formulation of the Storm Water Reuse program, it became apparent that the new city of Columbia, Maryland, offered several unique advantages as a study site for the systems analysis. The Storm Water Reuse concept is specifically directed to urbanized areas where water runoff can be clearly identified as a source of water pollution. In order to perform a systems analysis in such an area, the physical characteristics of the area must be known in some detail, including hydrologic factors, land uses, population densities, etc. Since the method of analysis was to be kept generalized wherever feasible, a study area was deliberately selected where gage data were not available for rainfall and runoff. The requirements that led to the close examination of Columbia, however, were those related to the planned demonstration of the concept following analysis.

Columbia is a completely planned new town being developed by Howard Research and Development Corporation of Baltimore, Maryland, an affiliate of the Rouse Company, on a 27 square mile site in Howard County, Maryland (Figure 2). Designed for a 1980 population of 110,000 people, Columbia is midway between Baltimore and Washington astride the busy Northeast Corridor. Construction started in 1966 and the first residents moved into the city in June 1967. By the first anniversary of the arrival of these residents, more than 5000 people were living in the community and construction was proceeding on schedule. The complete town will consist of nine villages with each village made up of a number of neighborhoods. A principal feature of the Columbia plan is the extensive use of water as a focal point for the activities of the community. The town site includes portions of the Little Patuxent, Middle Patuxent, and Patuxent river basins, and five man-made lakes totaling over 500 acres of water surface are scheduled. Two of these, Lake Kittamaquundi and Wilde Lake, are already in existence.

The availability of the complete Columbia plan permits the conduct of a systems study of the Storm Water Reuse concept in an area which is not yet fully developed. If this study is based on the planned characteristics of the area after development, a demonstration program can be conducted which will fully evaluate the results of the systems analysis while taking advantage of the reduced costs of concurrent construction. Since the entire area is being developed by one organization, coordination and cooperation are greatly enhanced. Another fortunate feature of Columbia is the integration of many different types of land use, including low to high density residential development, numerous types

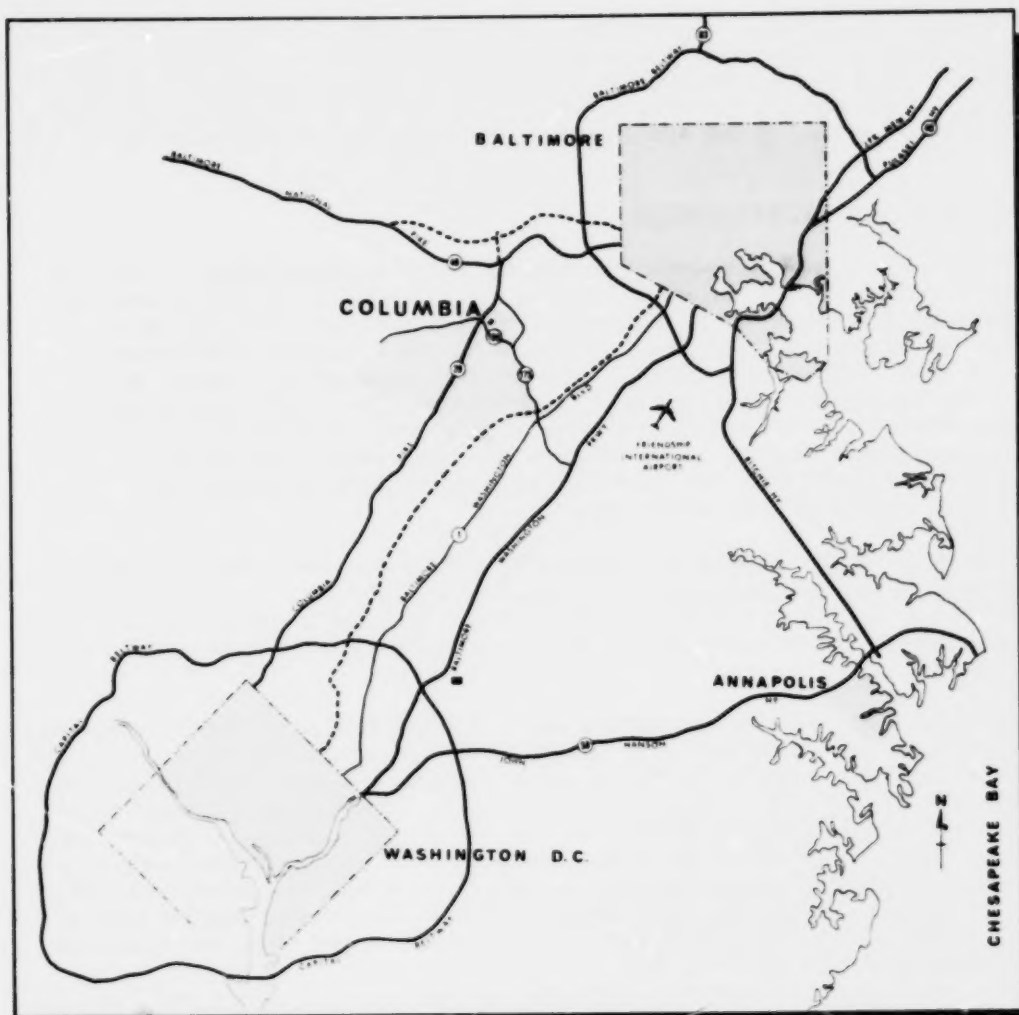


Figure 2. Location Map of Columbia, Maryland

of commercial uses, and various industrial uses. In some areas of Columbia, a wide variety of these uses can be found in a single watershed. As a result, the influence of various land uses on storm water quality and quantity as well as the accompanying possibilities for reuse can be evaluated in the systems study and subsequent demonstration.

WILDE LAKE WATERSHED

In order to simplify hydrologic aspects of the systems study as well as to create an easily definable water quality objective, it was decided to perform the study on the drainage area of one of the artificial lakes. Of the two lakes which would be available for the demonstration program, only one, Wilde Lake, had a drainage area substantially within the Columbia project. Furthermore, a great diversity of land use exists in the Wilde Lake watershed. Construction in the watershed is scheduled for completion before 1970, permitting concurrent construction of demonstration facilities, followed almost immediately by operation of these facilities in a fully developed area.

The Wilde Lake watershed consists of 1140 acres, or about 1.8 square miles. Other information concerning the lake and its watershed is listed in Table 1. Figure 3 is a map of the area, showing a number of sub-watersheds which were used in the hydrologic analysis reported in Section IV.

With the cooperation and assistance of Howard Research and Development Corporation, the Wilde Lake watershed was chosen as the site for the systems analysis effort of the Storm Water Reuse program, and the southwest quadrant of the watershed was identified as the tentative site of a first-phase demonstration facility following satisfactory completion of the systems analysis. To permit evaluation of various storm water reuse concepts in as many configurations as possible, the study watershed was divided into 22 sub-watersheds. These were delineated by natural ridge lines wherever possible, permitting the runoff from each area to be collected at a single point.

TABLE 1. THE WILDE LAKE WATERSHED, COLUMBIA, MARYLAND

Watershed Area	1,140	Acres
Planned Population	12,254	Persons
No. Dwelling Units	3,757	
Planned Land Use - Columbia		
Open Space	147.6	Acres
Single Family Res. - Low Density	13.3	Acres
Single Family Res. - Medium Density	185.8	Acres
Town houses	114.8	Acres
Garden Apartments	66.6	Acres
Mid-rise Apartments	4.9	Acres
Employment Centers	24.8	Acres
School Sites	37.7	Acres
Public Rights-of-way	115.8	Acres
Total - Columbia	711.3	Acres
Expected Land Use - Outside Columbia		
Open Space	194.5	Acres
Single Family Res. - Low Density	191.0	Acres
Public Rights-of-way	22.1	Acres
Total - Outside Columbia	407.6	Acres
Wilde Lake Water Surface Area	21.1	Acres
Estimated Maximum Capacity	48,200,000	Gallons
Fraction Impervious Construction in Watershed	0.22	

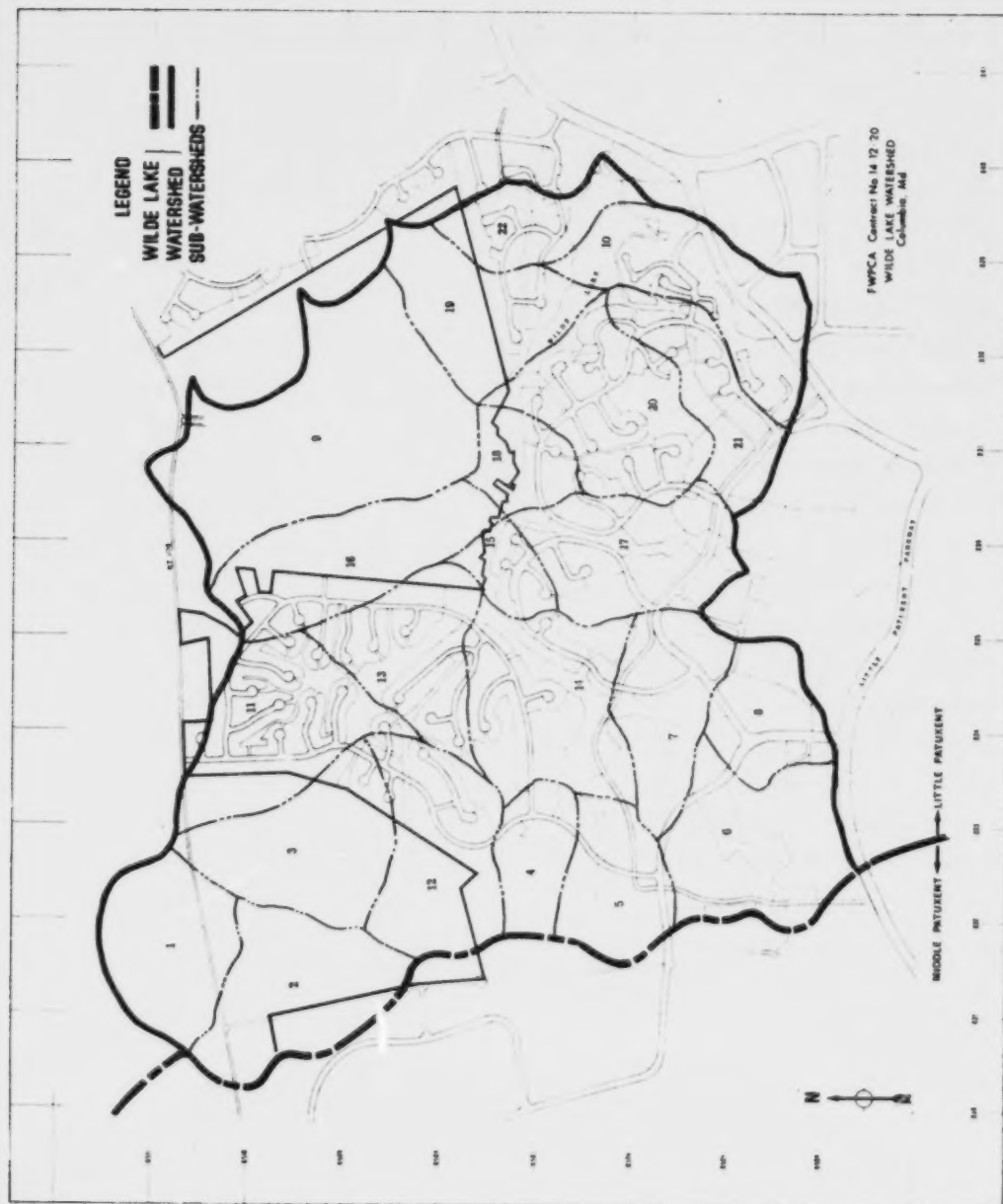


Figure 3. Wilde Lake Watershed - Sub-Watershed Map

SECTION IV

STORM WATER HYDROLOGY

As one of the initial phases of the Storm Water Reuse project, it was necessary to develop methods of adequately describing the storm water flow characteristics of the Wilde Lake drainage area. Two different types of hydrologic data were required:

A continuous daily runoff hydrograph from each of the sub-watersheds in the basin

Peak flows in each sub-watershed as a function of rainfall recurrence interval

The first of these was needed to estimate the total volume of water available for reservoir storage and reuse for any time interval, and the second to estimate sedimentation efficiency and system operation during large storms. Methods were required for obtaining both types of data without benefit of gaged stream flow records at Wilde Lake.

Fortunately, many methods exist for calculating peak flows from drainage areas lacking stream flow data. There are, however, few methods which can be applied to the development of a daily runoff hydrograph in an area where basic hydrologic data are unavailable. Since this problem was the first to be approached and required considerable effort for solution, it will be discussed in detail in the following sections.

AVAILABLE METHODS FOR CALCULATING CONTINUOUS RUNOFF HYDROGRAPHS

The method to be used in the derivation of a runoff hydrograph for the sub-watersheds of Wilde Lake had to satisfy the following requirements:

1. It had to provide a continuous hydrograph of daily runoff volumes for each sub-watershed and be applicable to the variety of impervious conditions of the watersheds.
2. It had to be a fairly simple method requiring a minimum of data input and a minimum amount of time spent in its development.
3. It had to be derivable without any basic hydrologic data except land use and rainfall.

For small rural watersheds such as Wilde Lake, there exists no standard simple method of providing a runoff model that would satisfy these requirements. To develop such a method would have required

more effort than could have been justified under this project since the selection of runoff values satisfied only a small part of the project goals. Therefore, several existing approaches to the solution of similar problems were examined with the purpose of selecting the best suited approach and modifying it as necessary.

Each of the methods that were examined has been used in the past to estimate runoff volumes and uses as one of the input parameters a rainfall record for the drainage area. The following approaches were considered.

Unit Hydrograph

This involves calculating for a drainage area the hydrograph that would be produced by one unit of "effective precipitation" (actual rainfall minus infiltration and other losses) occurring during a specific period of time (1). This unit hydrograph is then used to calculate the runoff hydrograph from any distribution of effective rainfall. On small drainage basins, the time intervals which would be required for rainfall input would be measured in minutes. The calculation of effective precipitation from actual precipitation would require a knowledge of infiltration losses during each rainfall at any time of the year.

Linear Storage Reservoir Models

This computer method, which has been employed by the U. S. Department of Agriculture (2), utilizes routings of effective precipitation through linear storage reservoirs with a built-in lag time. This method gives excellent results for individual storms, but also requires short-time interval inputs as well as infiltration knowledge to calculate effective precipitation.

Digital Simulation Models

Several models of this type exist with many variations. The basic problem with them, however, is that they require a great deal of detailed input information to describe the watershed.

Regression Models

This method takes a dependent variable and one or more independent variables and derives the coefficients relating the variables so that the variation between the observed dependent variable and the calculated dependent variable is at a minimum. Regression equations involving rainfall and runoff have been used in past hydrologic studies in areas where gaged data exist. However, it is possible to derive a regression equation for a gaged watershed and apply it to another ungaged area so long as the areas are physically and hydrologically similar.

It was decided that the adoption of some form of regression model would probably be the most fruitful approach. The procedure that was chosen was to select from the Maryland area another watershed that had the same physical and hydrologic conditions as the Wilde Lake area and had at least five years of rainfall and runoff data. A regression analysis could then be performed on this watershed and the results applied to Wilde Lake.

The advantage of this approach is that a detailed knowledge of infiltration and soil conditions is not required since the overall effects of such parameters are indicated by the regression coefficients. The regression model does not have to present an exact cause-effect relationship between variables as long as its predictive ability is good and the variable relationship is reasonable.

DEVELOPMENT OF REGRESSION MODELS

After examining the available runoff data in the Maryland area, only one watershed was located having physical characteristics similar to those of Wilde Lake. This watershed, on the West Branch of Herring Run in Baltimore County, is of the same approximate size and shape, has the same topographic features, and is located in the same geologic zone as the Wilde Lake drainage area. For these reasons it is ideally suited for the derivation of a rainfall-runoff model which could then be applied to Wilde Lake with modifications to account for varying percentages of imperviousness in the sub-watersheds.

A number of regression models were tested including models in which the effect of antecedent precipitation on runoff and the seasonal variations of base flow were considered. The model found to provide the best correlations with actual data consisted of a third order polynomial shown in Table 2.

TABLE 2. FINAL REGRESSION EQUATION COEFFICIENTS

	$Q_i = A + BR_i + CR_i^2 + DR_i^3$				(4)	
	Regression Coefficients				Correlation	
	A	B	C	D	R	R ²
Yearly	0.5247	3.016	1.187	-0.168	0.858	0.736
Seasonal						
Winter	0.885	3.449	0.808	0.099	0.756	0.572
Spring	0.625	2.044	1.917	-0.419	0.874	0.764
Summer	0.269	1.849	1.969	-0.260	0.974	0.949
Fall	0.353	1.297	4.591	-1.389	0.927	0.859
R_i	= rainfall in inches/day					
Q_i	= runoff in cfs-days/mile ² -day					

APPLICATION OF THE REGRESSION MODEL TO WILDE LAKE

Although the Herring Run and Wilde Lake areas are similar in most respects, the sub-watersheds of Wilde Lake will have a variety of impervious percentages after development. Past studies of the effects of urbanization on runoff (3) have shown that an increase in impervious cover will decrease the lag time between peak rainfall and runoff, increase the peak runoff rate, and increase the runoff yield per unit area. It is only this last effect which will affect the derived runoff model in this case, since the areas are too small for lag time changes to be important on a daily flow basis, and the peak flows will only affect the pipe sizings.

It was therefore necessary to develop a procedure for modifying the model to take into account a full range of impervious conditions. This involves several assumptions:

1. For the Herring Run area, which is 11.9 percent impervious, equations can be derived representing unit flow from both the completely pervious and completely impervious portions of the basin.
2. The yield from the completely impervious portions will be less than 100 percent of the rain that falls on them, since some water will flow from impervious surfaces to pervious surfaces where it will be lost to infiltration.
3. While this percentage will actually vary, an average value can be calculated and used in the equations.

These assumptions were used in the derivation of the following equation, which is fully described in Appendix A:

$$Q_i = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right) \left[(B - KXI)R_i + CR_i^2 + DR_i^3 \right] \quad (5)$$

where:

- Q_i = runoff/unit area from any Wilde Lake watershed on the i^{th} day
- K = conversion factor between in. / day and cfs/mi² = 26.88
- X = fraction of rainfall actually running off impervious areas
- I = fraction imperviousness (Herring Run = 0.119)
- I' = fraction imperviousness of any Wilde Lake watershed
- R_i = rainfall on the i^{th} day
- A, B, C, D = regression coefficients derived previously

The only unknown in this equation is X, the percentage of runoff from impervious areas. To obtain a value, the results of a study of effects of urbanization on runoff from selected Texas watersheds were used (3). By relating increased runoff to increased imperviousness on the same watershed, the following equation was derived:

$$X = \frac{Q_i}{R} \left[\frac{1}{I} + \frac{1-I}{I(1+100I)^{0.339}} \right] \quad (6)$$

where:

Q_i = runoff volume (as calculated by the equation given in Appendix A) for an area with an impervious fraction of I

R = rainfall volume

By taking individual storm data and calculating Q_i , a value of X can be calculated for each storm, and these values can be averaged over all storms. The value of $\bar{X}$ thus obtained is:

$$\bar{X} = 0.8536$$

Using this value in the equation for Q_i , the final equation for runoff on the Wilde Lake watersheds becomes:

$$Q_i = A + R_i \left[23.08I' + \left(\frac{1-I'}{0.88} \right) (B-2.77) \right] + R_i^2 \left[C \left(\frac{1-I'}{0.88} \right) \right] + R_i^3 \left[D \left(\frac{1-I'}{0.88} \right) \right] \quad (7)$$

Separate equations for each sub-watershed and for each season can be derived by using the appropriate values of I' and the regression coefficients A, B, C, and D, and from these equations the continuous runoff hydrograph can be calculated using a daily rainfall input.

To illustrate the actual effects of imperviousness on the runoff yield per unit area, an equation can be developed giving the percent increase in unit yield of an area with a fraction imperviousness of I' over the natural area (0 impervious fraction).

$$P = \frac{100I' \left[KX - B - CR_i - DR_i^2 \right]}{\frac{A(1-I)}{R_i} - KXI + B + CR_i + DR_i^2} \quad (\text{Appendix A}) \quad (8)$$

where P = percent increase in yield over the natural state. Figures 4 and 5 show, respectively, P versus I' for different values of R_i and P versus R_i for different values of I' . The results of the Texas study on effects of urbanization are also indicated in Figure 4. It can be seen that for every value of I' , the precipitation values between about 0.2 inch and 1.0 inch have the greatest effects on P. This would be expected

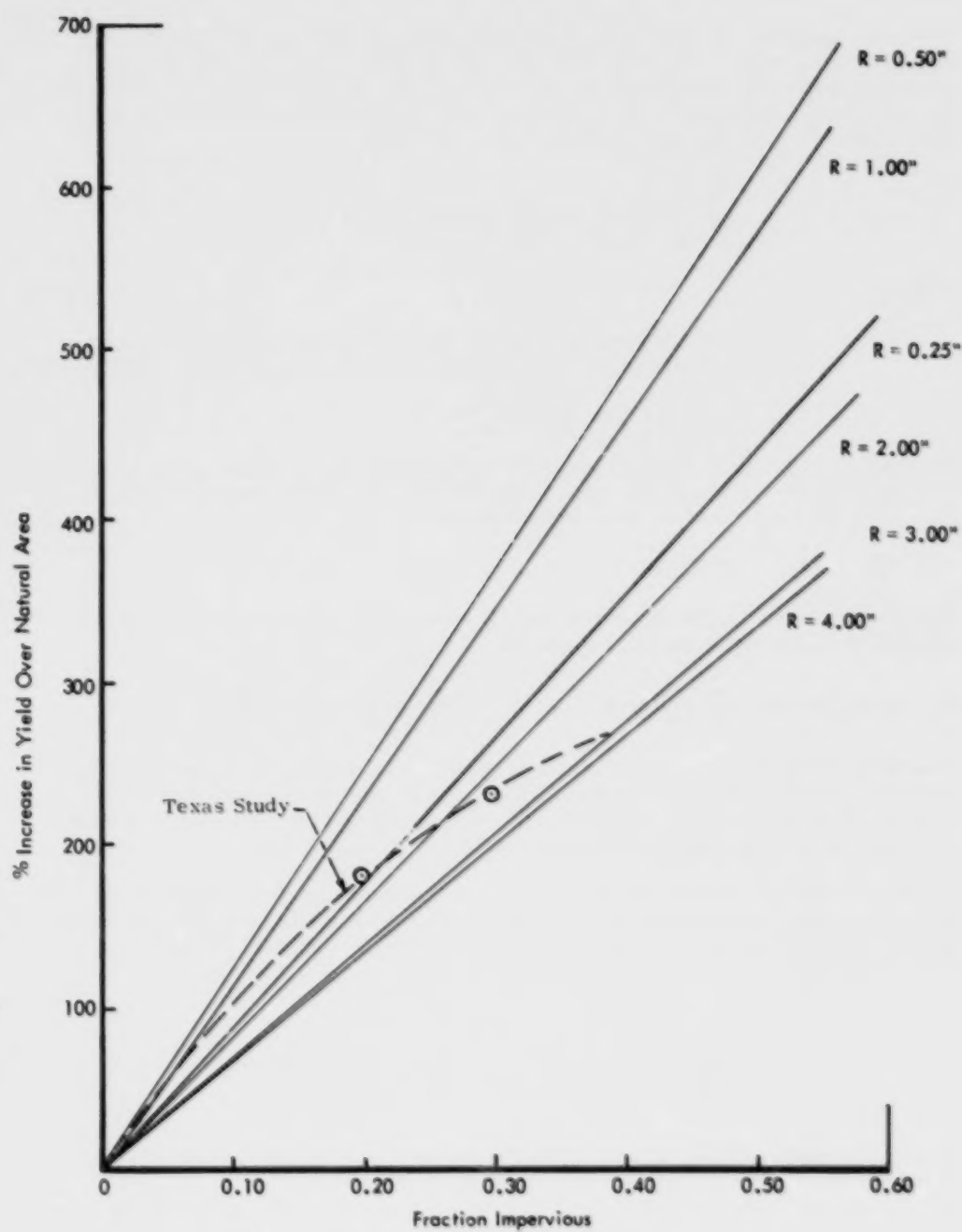


Figure 4. Increased Yield vs. Fraction Impervious

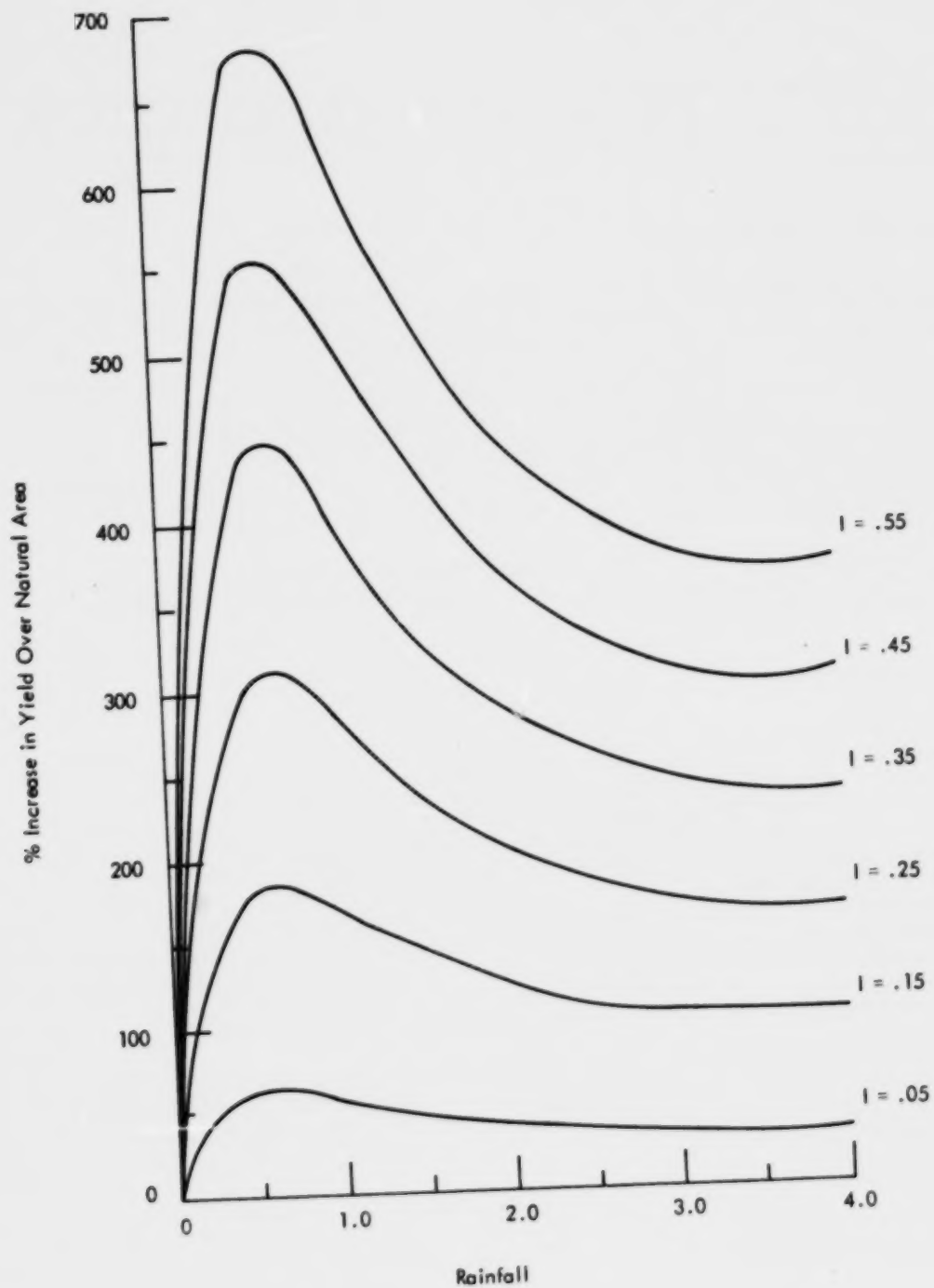


Figure 5. Increased Yield vs. Rainfall Volume

since during small rainstorms the pervious areas would contribute very little runoff, and during heavy rains, the pervious areas would eventually contribute large volumes because the infiltration capacity would be exceeded.

SENSITIVITY ANALYSIS

As one method of testing the usefulness of the final modified regression equation, a sensitivity analysis was made of the equation coefficients. Such an analysis indicates how sensitive the calculated values of runoff are to minor variations in these coefficients and also provides a measure of the reliability of the equation over the range of rainfall values. To perform the type of sensitivity analysis required, the regression equation was partially differentiated in the terms of each coefficient, and the resulting differential equations were used to calculate the change in runoff as a function of the change in coefficients. This was done for both the upper and lower limits of the imperviousness parameter.

It was determined that the final regression equation is not abnormally affected by coefficient changes, and throughout the rainfall range the equation remains stable.

STORM HYDROGRAPH AND PEAK FLOW MODEL DEVELOPMENT

In order to calculate the expected peak flows for a storm of any recurrence interval, a method was desired which could use the rainfall intensity-duration-frequency curves for Howard County as a basic input. Although the standard procedure for this type of calculation is to use the Rational Method with an appropriate runoff coefficient for each watershed, a more sophisticated approach which would provide more detailed information was selected. This approach, mentioned previously, involves the routing of a rainfall hyetograph through a linear storage reservoir model to obtain a complete storm hydrograph. It has been used with great success by the U. S. Department of Agriculture in analyzing storm runoff from small agricultural watersheds (2), and it required only minor changes to be adapted to this study.

As the first step in this analysis, the Howard County rainfall curves were fitted to an equation relating the intensity of rainfall, the duration of rainfall, and the return period of the storm. This equation, in a standard form, is:

$$i = \frac{200T^{0.176}}{(t + 25)^{1.1}} \quad (9)$$

where:

- i = intensity (inches/hour)
- T = return period (years)
- t = duration of rainfall (minutes)

Using this equation, rainfall hyetographs for a 60-minute storm with return periods on one, two, five, and ten years were synthesized. These hyetographs were developed with the following conditions:

1. For each storm, the intensity of rainfall for any duration from five minutes to 60 minutes is identical to the intensity calculated by the rainfall equation.
2. The hyetograph is approximately symmetric about the peak intensity.
3. The minimum time interval for calculating intensities is five minutes.

In addition to these conditions, the 60-minute storm volume is only slightly less than the volume of a 24-hour storm of the same return period, and therefore is analogous to the daily rainfall and runoff volumes used in the regression analysis.

The linear storage reservoir model, which calculates the runoff hydrograph, requires effective precipitation as an input. Since effective precipitation is simply the rainfall hyetograph minus any losses due to infiltration or surface storage, a method was needed to alter the hyetograph to include these losses. This was done by using an infiltration equation for the losses on the pervious portion of a watershed and by assuming the rainfall on the impervious portion is entirely available for runoff. The chosen infiltration equation was one originally derived by Horton (4) which had been used in a Johns Hopkins University study of the rainfall in runoff processes (5) and for which values of infiltration coefficients had been calculated for the Baltimore area. The steps involved in changing the original hyetograph are summarized in the following equations, which were applied to the hyetograph for each interval of Δt :

$$P_{Per} = P_{Imp} - f = P_{Imp} - \left[f_o + (f_c - f_o)(1 - e^{-at}) \right] \quad (10)$$

where:

- f = infiltration loss = $f_o + (f_c - f_o)(1 - e^{-at})$
- f_o = initial infiltration rate (inches/hour) = 2.5

$$\begin{aligned}
 f_c &= \text{final infiltration rate (inches/hour)} = 1.5 \\
 a &= \text{exponential decay constant (hour}^{-1}\text{)} = 0.7 \\
 P_{\text{Per}} &= \text{effective rainfall on pervious areas} \\
 P_{\text{Imp}} &= \text{rainfall on impervious areas} = \text{original hyetograph} \\
 P_{\text{Per}} &= 0 \text{ when } f > P_{\text{Imp}} \quad (11)
 \end{aligned}$$

$$P_{\text{Eff}} = P_{\text{Imp}} (I) + P_{\text{Per}} (1-I) \quad (12)$$

where:

$$\begin{aligned}
 P_{\text{Eff}} &= \text{final effective precipitation hyetograph} \\
 I &= \text{fraction imperviousness of the watershed}
 \end{aligned}$$

The resulting effective precipitation hyetograph has been applied as input to a linear storage reservoir model of the runoff process. This model uses the following continuity equation as its basis:

$$I_i = \frac{q_1 + q_2}{2} + \frac{S_2 - S_1}{\Delta t} \quad (13)$$

where:

$$\begin{aligned}
 I_i &= \text{average inflow rate for time period } \Delta t (\text{cfs}) \\
 q &= \text{outflow rate (cfs)} \\
 S &= \text{storage volume (feet}^3\text{)} \\
 \Delta t &= \text{time increment between times 1 and 2, seconds}
 \end{aligned}$$

The effect of storage on the outflow depends on the physical properties of the entire drainage area and on the volume of storage. Normally the storage will impart a time delay to the outflow with respect to the inflow into the basin. This concept is expressed by the equation:

$$I_i - q = M \frac{\Delta q}{\Delta t} \quad (14)$$

where M is the basin storage coefficient, which may be considered to be equal to the lag time (2).

In order to obtain the best estimates of peak flow, timing, and hydrograph shape, investigations have shown that two routings of the rainfall

through half of the indicated storage are required, where the output from the first reservoir becomes the input to the second.

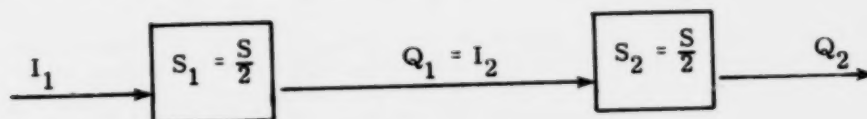


Figure 6. Routing Method - Linear Storage Reservoir Model

From the two preceding equations and the double-routing technique, a single equation can be derived to take effective precipitation as the input to the first reservoir and calculate the output from the second reservoir.

This final equation for routing the effective precipitation is:

$$Q_i = \left(\frac{\Delta t}{m + \Delta t} \right)^2 \left(P_{\text{Eff}_i} + 2P_{\text{Eff}_{i-1}} + P_{\text{Eff}_{i-2}} \right) + 2 \left(\frac{m - \Delta t}{m + \Delta t} \right) Q_{i-1} - \left(\frac{m - \Delta t}{m + \Delta t} \right)^2 Q_{i-2} \quad (15)$$

where:

$$\begin{aligned} Q_i &= \text{runoff in cfs/acre during the } i^{\text{th}} \text{ time interval} \\ m &= \text{lag time (minutes)} \\ \Delta t &= \text{time interval (minutes)} \end{aligned}$$

Using a computer, the effective precipitation hyetographs for each storm return period, and for a range of imperviousness from 0.1 to 0.5, were used as input to the routing equation. Various lag times from 4 minutes to 20 minutes were assumed and a total of 48 hydrographs representing each combination of return period, imperviousness, and lag time was obtained as output.

These data were then reduced by calculating for each hydrograph the maximum average flow for durations from five minutes to 60 minutes, or in equation form:

$$Q_d = \text{Max} \left(\sum_d Q/d \right), \text{ where } d = \text{duration in minutes} \quad (16)$$

The resulting values of Q_d , plotted against duration (for every combination of lag time, imperviousness, and return period), provided a set of curves that are analogous to the rainfall intensity-duration-frequency

curves. The evolution of these runoff curves is summarized in graphical form in Figure 7, and the effects of all the parameters on Q can be seen in Figures 8 through 10.

In order to more easily use the data that these curves represent, an equation was graphically fitted to all of the points for durations up to 30 minutes. This equation related all of the parameters, eliminating the need for interpolation between the original curves and allowing a rapid determination of peak flows for any sub-watershed. The equation which follows has a multiple correlation coefficient of 0.98 when compared to the original data points.

$$Q_d = 8T^{0.301}(m+3)^{-0.732} - 16T^{0.246}(m+10)^{-2.200}d + 0.955T^{0.033}I(2.482 - 0.129m + 0.010m^2 - 0.00033m^3) \quad (17)$$

where:

m	=	lag time (minutes)	$4 \leq m \leq 20$
d	=	duration (minutes)	$5 \leq d \leq 30$
T	=	return period (years)	$1 \leq T \leq 10$
I	=	impervious fraction	$0.1 \leq I \leq 0.5$

By applying this equation to the sub-watersheds, storm flow determinations could be made for any storm up to 10 years in magnitude, and the results of these determinations and their application to sedimentation requirements are described in a subsequent section.

As a final application of this storm hydrograph analysis, the volumes of the individual hydrographs were calculated and plotted against imperviousness and return period. These runoff volumes represent approximately the maximum runoff volumes of a 24-hour storm for each return period since the original 60-minute rainfall hyetograph volume was almost identical to the 24-hour rainfall volume. A graph of storm volumes is shown in Figure 11.

COMPARISON OF REGRESSION MODEL AND STORM HYDROGRAPH MODEL

Since two entirely different techniques were used to analyze storm water runoff, a comparison was made of the results of the two. Because both methods ultimately calculated the volumes of runoff, storm volumes were chosen as the basis for comparison.

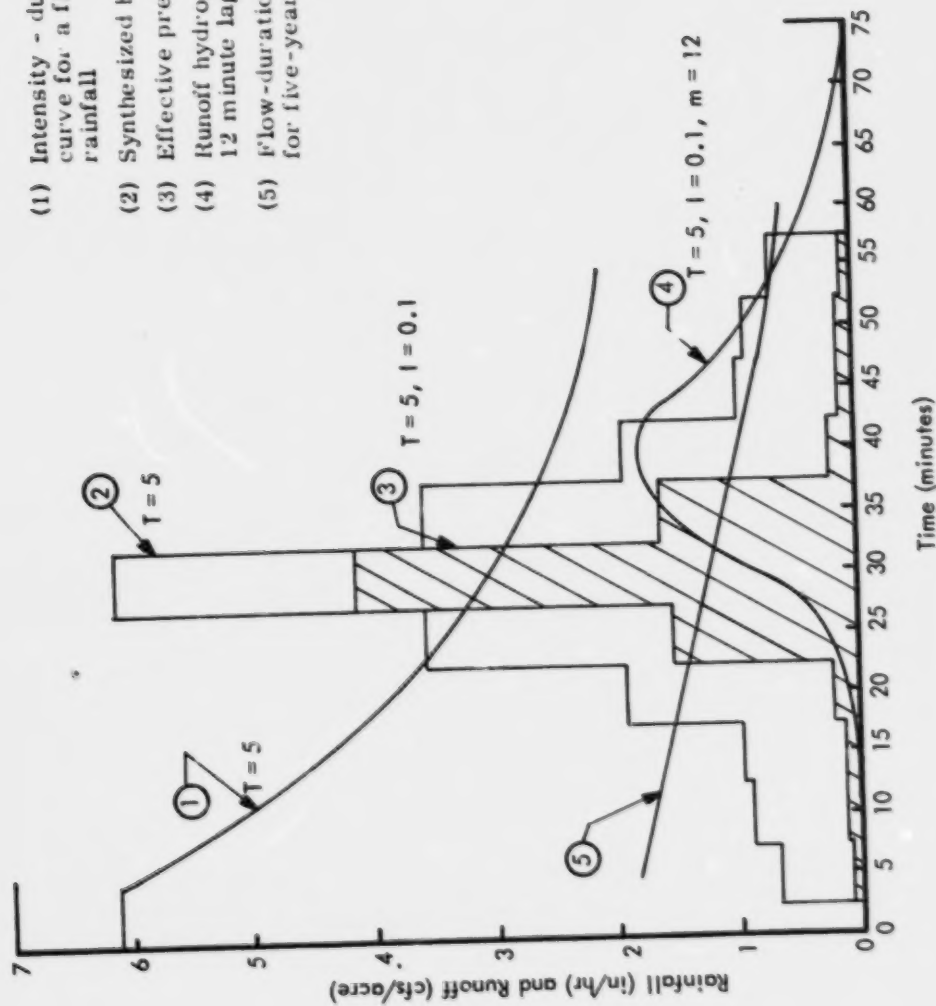


Figure 7. Derivation of Flow-Duration-Frequency Curve for Five-Year Storm Runoff

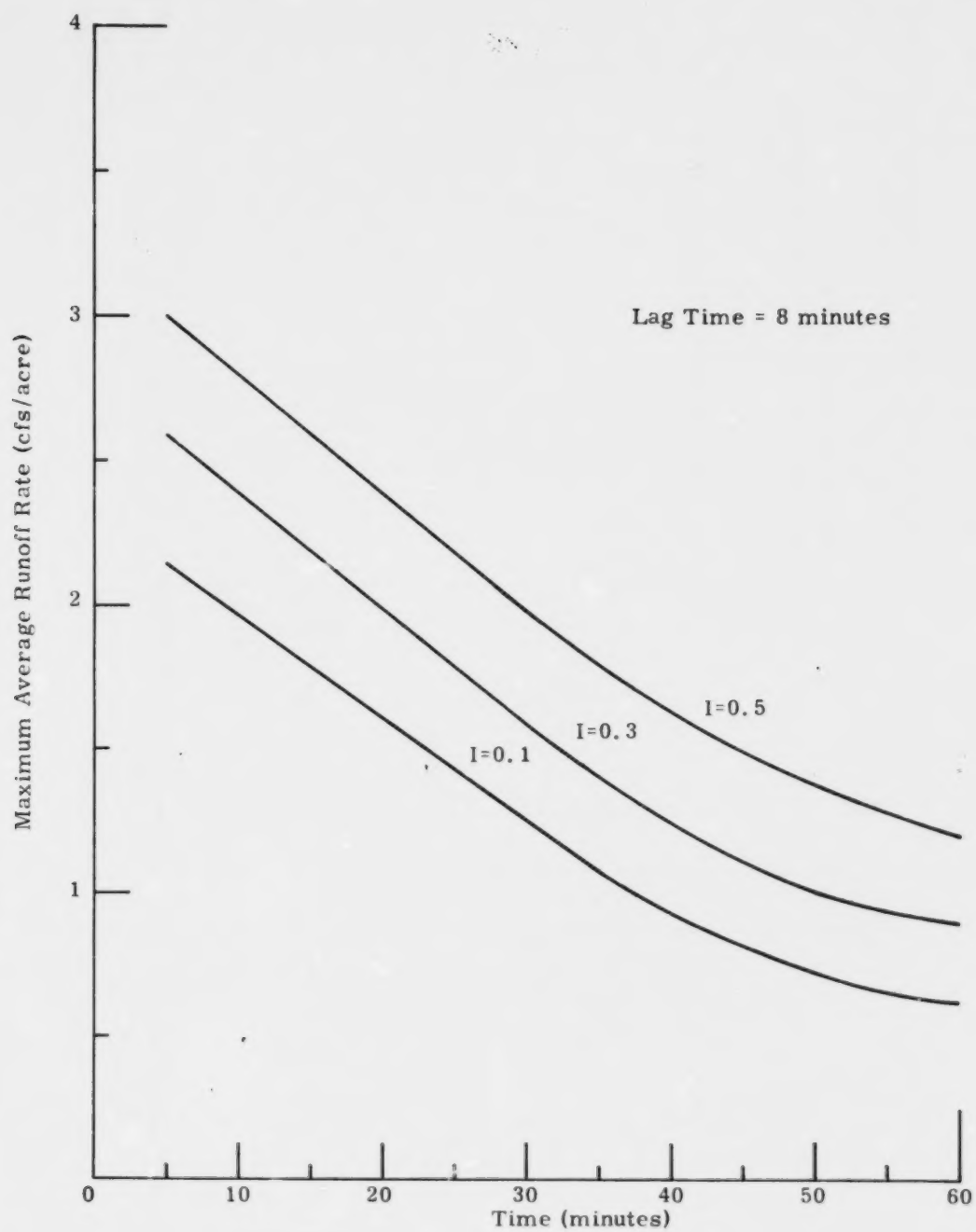


Figure 8. Effect of Impervious Fraction (I) on Five-Year Storm Runoff

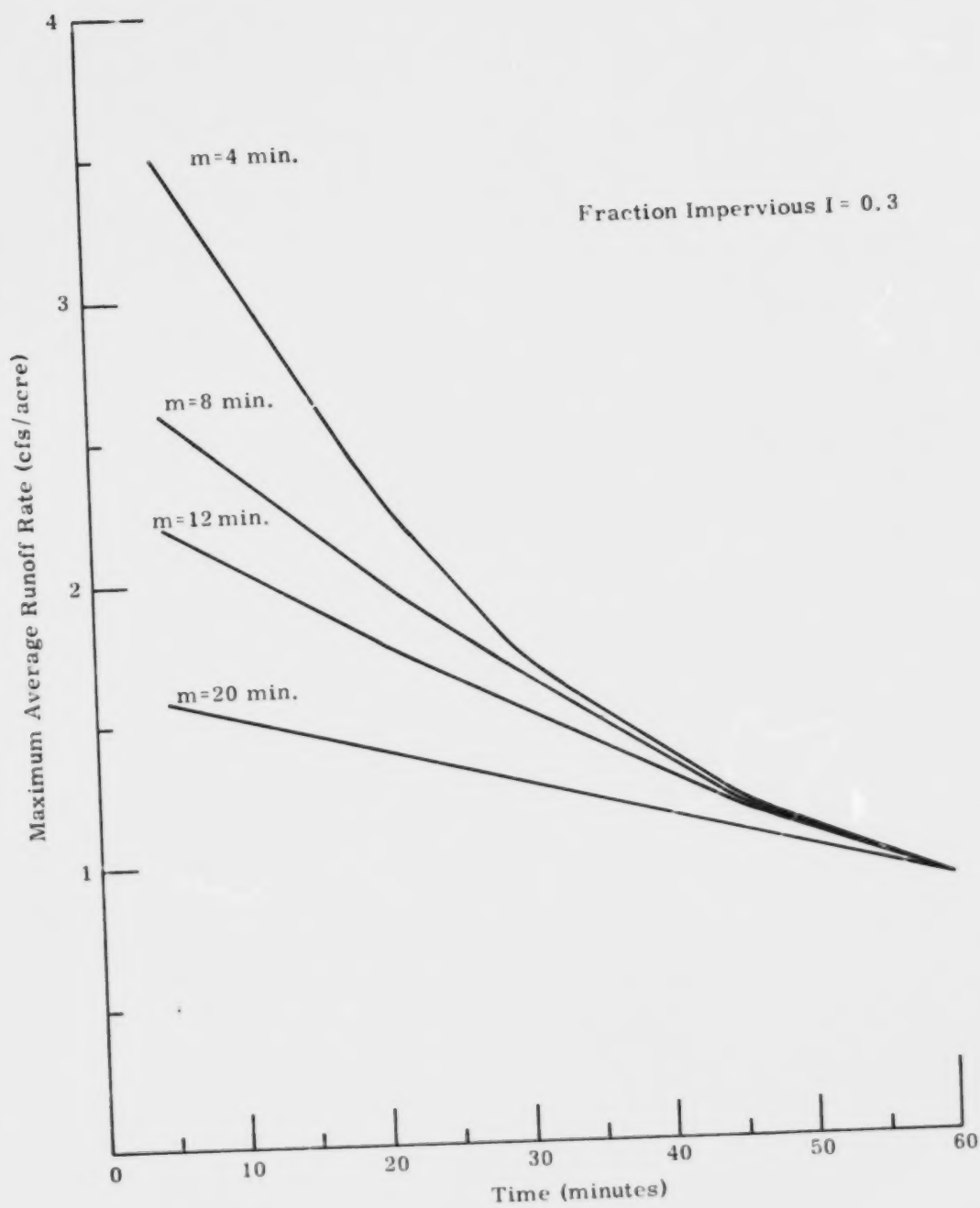


Figure 9. Effect of Lag Time (m) on Five-Year Storm Runoff

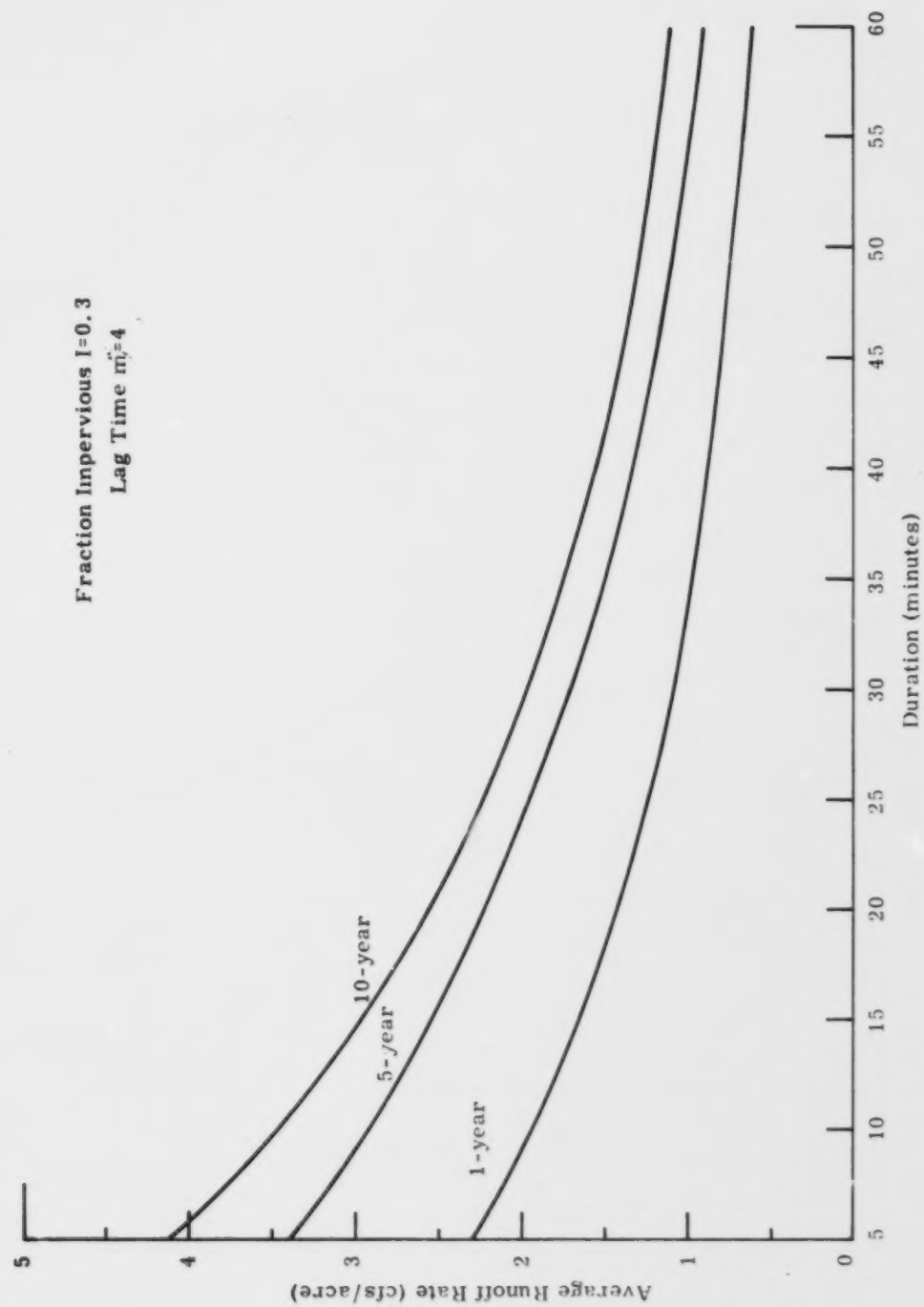


Figure 10. Average Runoff Rate vs. Storm Duration

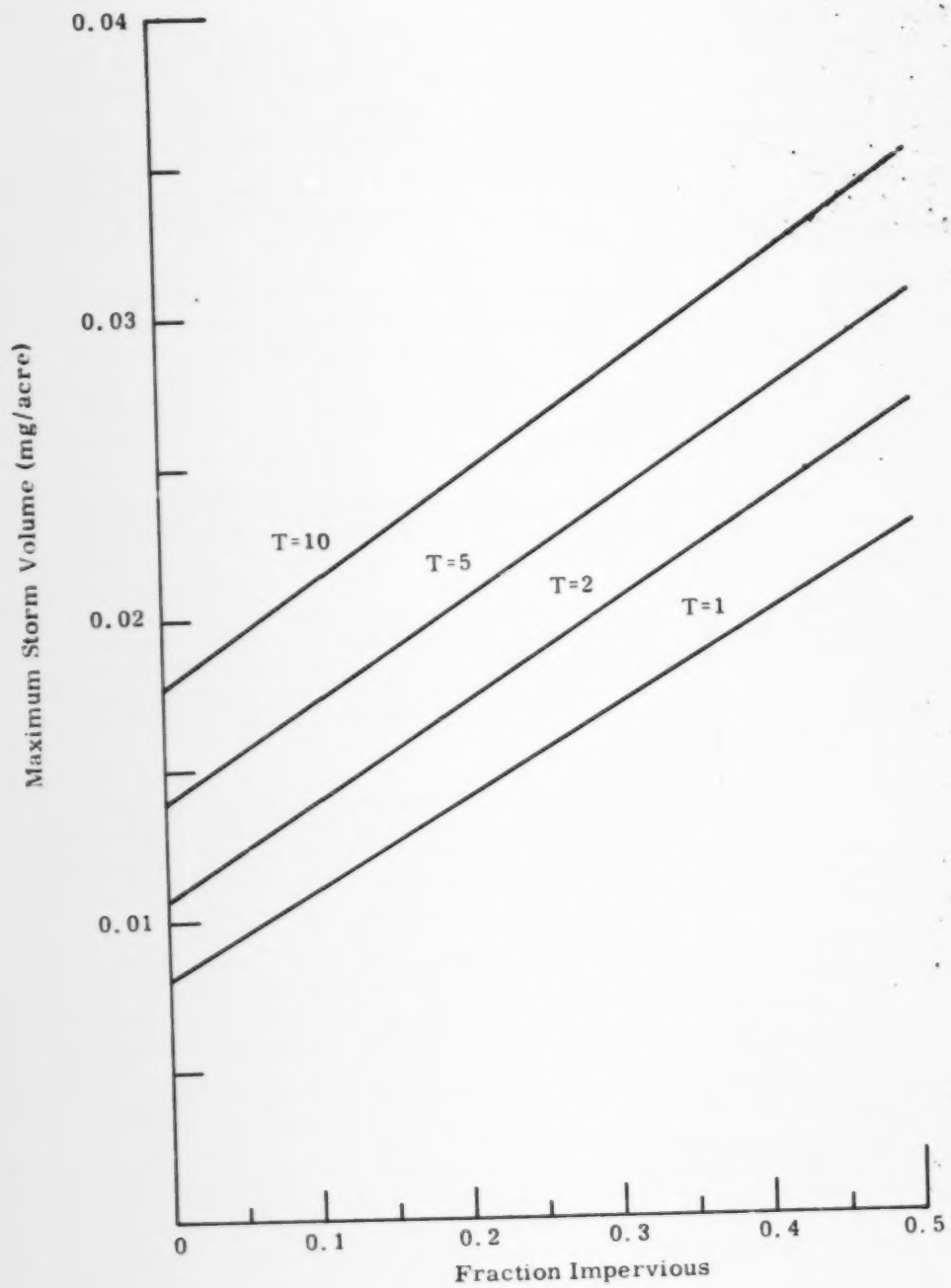


Figure 11. Maximum Storm Volumes

A five-year computer calculation of daily flows for the Wilde Lake watershed, as determined by the regression model, was examined and a frequency distribution of the runoff was plotted. This was then converted to a plot of daily flow versus return period as shown in Figure 12. To calculate the volumes of storms using the hydrograph approach, curves similar to those in Figure 11 were employed, using a fraction imperviousness of 0.22. This provided storm volumes for storms with return periods of one, two, five, and ten years, which could be compared to the corresponding volumes in Figure 12.

A plot of storm volumes calculated by both methods is given in Figure 13. It can be seen that the different techniques gave approximately the same results with a maximum difference of only eight percent for a two-year storm.

INPUT TO THE SYSTEM ANALYSIS

In order to use the final regression equation (Equation 7) to calculate continuous runoff hydrographs for each of the Wilde Lake sub-watersheds, it was necessary to first determine the fraction imperviousness of each watershed. Since land use data were readily available for the Columbia project, a tabulation was made of the acreage that was to be devoted to each type of land use in every sub-watershed. A factor was applied to each use to represent the percent imperviousness estimated for that type of use. By multiplying the impervious factor by the number of acres and summing for all land uses, the total number of pervious and impervious acres could be obtained.

For the portions of watersheds that were situated outside the Columbia boundaries, a similar procedure was followed. In this case, either existing land uses or zoning maps were used to tabulate the land uses. The same impervious factors were applied to the non-Columbia acreage as were used for similar uses in the Columbia project.

Figure 14 is an actual tabulation of land uses, residence units, population, and imperviousnesses for sub-watershed number 17. The same tabulation was prepared for all other watersheds and the overall imperviousness (shown in the last line of Figure 14) was used as I in Equation 7.

The only additional data required to calculate runoff hydrographs were daily precipitation for the Wilde Lake area. After examining monthly precipitation data from four rain gages in the vicinity of Columbia, it could be seen that the Friendship Airport weather station data agreed most closely with the monthly rainfall calculated for Columbia by the Thiessen method. Therefore, five years of daily rainfall data were obtained from Friendship Airport to be used as input to the regression model.

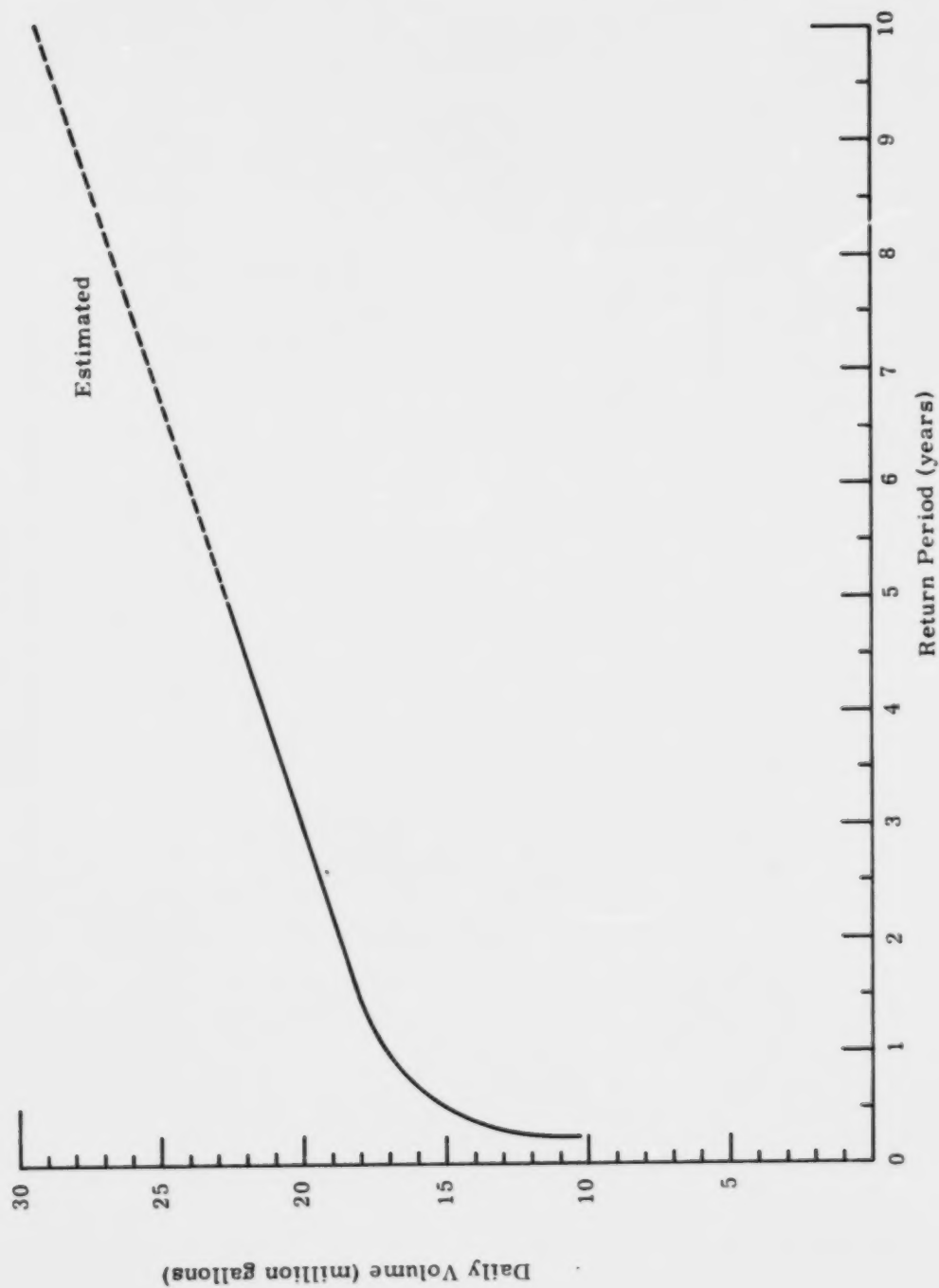


Figure 12. Calculated Daily Volume vs. Return Period - Wilde Lake

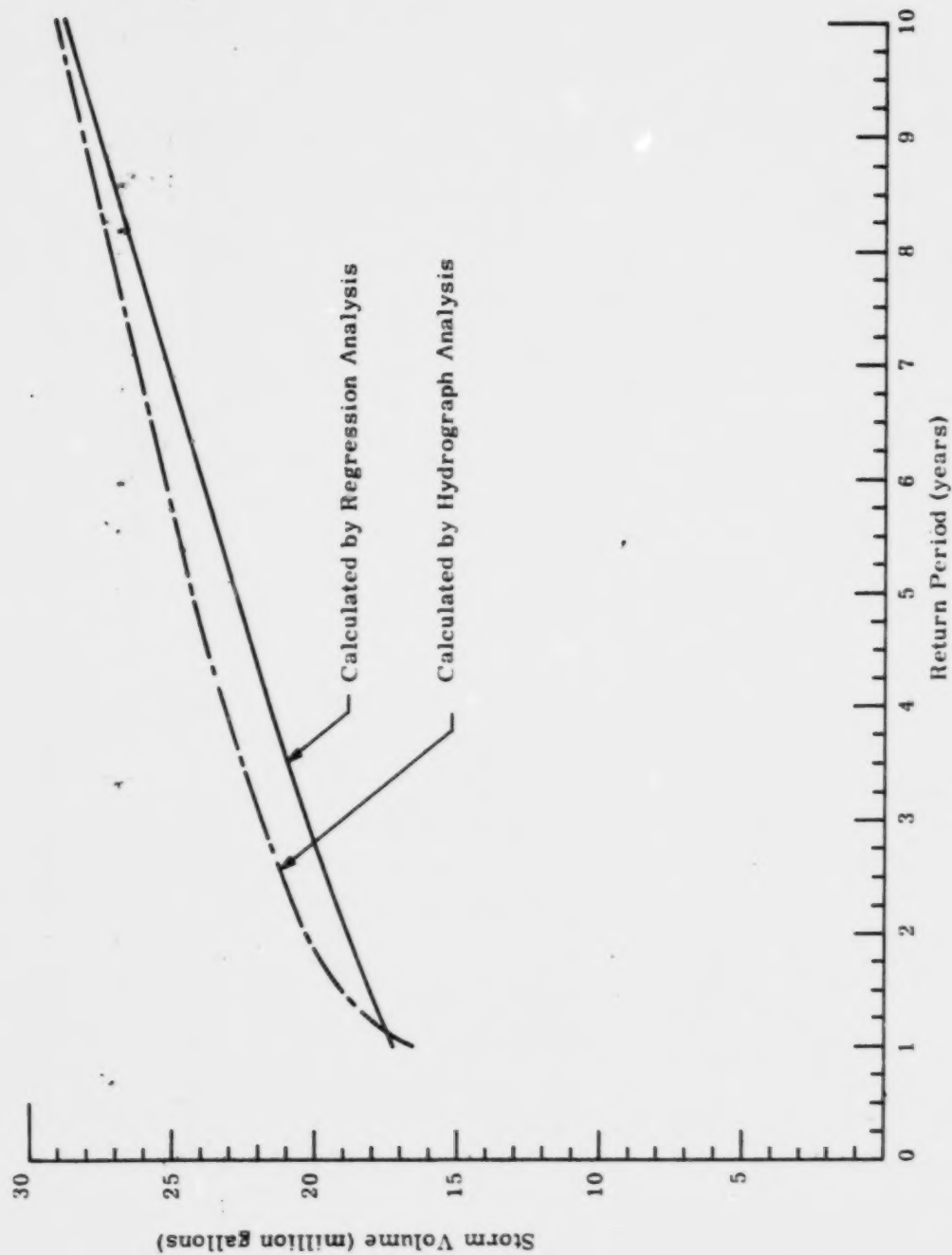


Figure 13. Calculated Daily Volume vs. Return Period -
Wilde Lake Watershed

Project 3519

SUB-DRAINAGE AREA
DATA SHEET

Coll. Area No. 17

	LAND USE	AREA (Ac.)	UNITS/ Ac.	RESID. UNITS	POPUL. UNIT	POPUL.	% IMPERV.	IMPERV. AREA (Ac.)
COLUMBIA PROJECT	OPEN SPACE	10.7					0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Single Family - Medium Density	12.5	4.0	50	4.0	200	16.0	2.0
	Town-Houses	4.9	11.0	54	3.5	189	33.0	1.6
	Garden Apartments	6.8	17.0	116	2.5	290	20.0	1.4
	Mid-Rise Apartments	3.6	28.0	101	2.5	253	22.0	0.8
	Employment Centers	7.4					100.0	7.4
	School Sites	2.5					18.0	0.5
	Public Rights-of-way	9.5					85.0	8.1
	OPEN SPACE	0.1					0.0	
NON-COLUMBIA	Single Family - Low Density		1.2		4.0		7.0	
	Public Rights-of-way						8.5	
	COLUMBIA TOTAL	57.9			321	932	37.7	21.8
	NON-COLUMBIA TOTAL	0.1						
	AREA TOTAL	58.0			321	932	37.6	21.8

Figure 14. Work Sheet - Typical Sub-Watershed Land
Use Tabulation and Imperviousness Factor Calculation

These data were first altered to more closely represent the long-term precipitation record at Friendship Airport. To perform the alteration, a simple normalization technique was used which preserved the historical time distribution of rainfall, but which changed the daily volume of rainfall on a random basis. This alteration eliminated the extremely dry or wet months, which on the average would be unlikely to occur in a given five-year period, without eliminating most daily extremes or greatly changing yearly totals. The procedure used involved the following data input:

1. A set of 30 monthly rainfall totals to represent typical monthly rainfalls expected. These were obtained from a 50-year distribution of monthly precipitation at the Baltimore Customs House. These values are shown in Table 3.
2. The continuous daily rainfall record for Friendship Airport expressed in inches for the five-year period 1960 through 1964.

TABLE 3. TYPICAL MONTHLY RAINFALL TOTALS
BALTIMORE CUSTOMS HOUSE, 1900-1950

<u>Month</u>	<u>Total Monthly Precipitation (inches)</u>	<u>Month</u>	<u>Total Monthly Precipitation (inches)</u>
1	0.40	16	3.05
2	0.69	17	3.21
3	0.92	18	3.41
4	1.18	19	3.60
5	1.40	20	3.80
6	1.52	21	4.01
7	1.71	22	4.28
8	1.85	23	4.50
9	2.00	24	4.80
10	2.15	25	5.10
11	2.29	26	5.50
12	2.41	27	6.00
13	2.59	28	6.65
14	2.71	29	7.45
15	2.90	30	10.70

A computer program was used to take each month's daily rainfall data and sum these data to give the total monthly rainfall. This monthly value (M_i) was compared to the set of 30 typical monthly values and the month that was closest to M_i numerically was chosen. (This can be called T_i .) The difference between M_i and T_i was calculated and M_i was increased or decreased by this amount so that it equaled T_i . The adjustment was done in the following manner:

1. If M_i was greater than T_i , one day during the month was chosen at random and 0.01 inch of rainfall was subtracted. (If that day

had no rainfall, then another day was chosen.) This was done repeatedly until $M_i = T_i$.

2. If M_i was less than T_i , one day during the month was chosen at random and 0.01 inch of rainfall was added. (Once again, only days on which rainfall actually occurred were changed.) This was done repeatedly until $M_i = T_i$.

As a numerical example of this, assume that for January 1962 a total of 5.02 inches of rain fell according to the sum of the first 31 daily volumes. It is found by comparison to the 30 typical monthly values that the typical month which is numerically the closest to 5.02 inches is one with 5.10 inches of rain. This value therefore represents T_i .

$$T_i - M_i = 0.08 \text{ inch} \quad (18)$$

Since M_i is less than T_i , the 0.08 inch must be added to the daily rainfall during the month of January 1955. A day on which rainfall occurred would be picked at random and 0.01 inch would be added to the rainfall data for this day. This procedure would have to be done 8 times until the original 5.02 inches of January rainfall would equal 5.10 inches.

This procedure was followed for each of the 60 months in the five-year record and the adjusted daily precipitation was then used as the input to the regression model. The computer was used to output daily runoff values (neglecting base flow) for each sub-watershed and combination of watersheds. A typical year's output for one of the sub-watersheds, including coefficients, is shown in Figure 15.

It should be noted that the five years of rainfall data, when compared to the long-term record, consist of one very wet year, three dry years, and one average year. The occurrence of below average precipitation during three years should provide a conservative input to an economic study of storm water reuse.

STORM VOLUMES

To provide the estimates of one-year storm volumes for different watersheds, the results of the hydrograph analysis were employed. For every sub-watershed, the values of imperviousness and size were used to obtain from Figure 11 the total volumes of a one-year storm. These values were then tabulated and are presented in Table 4. The application of these volumes to the sizing of reservoir ponds and sedimentation facilities will be discussed in a subsequent section.

GAGING AND SAMPLING

As a part of this program, a gaging and sampling station was established in the Wilde Lake watershed. The results of this work are contained in Appendix B.

AREA = 0.1078 SQ. MILES

RUN NO. = 6

REGRESSION COEFFICIENTS

	A	B	C	D	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
JAN	0.0	6.6134	0.6777	0.0827	0.69	0.05	0.09	0.01	0.02	0.09	0.26					
FEB	0.0	6.6134	0.6777	0.0827		0.95	0.06			0.07						
MAR	0.0	6.6134	0.6777	0.0827		0.10	0.04				0.36	0.42				
APR	0.0	5.4347	1.6078	-0.3519			0.70	0.66	0.12	0.10	0.02	0.15	0.03	0.15		
MAY	0.0	5.4347	1.6078	-0.3519					0.32	0.77	0.01	0.15	0.09	0.06		
JUNE	0.0	5.4347	1.6078	-0.3519					0.44	0.02	0.21	0.06				
JULY	0.0	5.2710	1.6515	-0.2183					0.04	1.29	0.08	0.04				
AUG	0.0	5.2710	1.6515	-0.2183					0.04	0.02	0.90	0.04				
SEPT	0.0	4.8080	3.8510	-1.1654					0.16	0.02	0.07					
OCT	0.0	4.8080	3.8510	-1.1654												
NOV	0.0	4.8080	3.8510	-1.1654												
DEC	0.0	4.8080	3.8510	-1.1654												
1																
2																
3																
4																
5																
6																
7																
8																
9																
10																
11																
12																
13																
14																
15	0.53															
16	0.05															
17		0.32	0.39													
18	0.02	0.18														
19	0.05															
20																
21		0.12	0.54	0.09												
22	0.03	0.29	0.12													
23		0.46														
24		0.34														
25	0.27	0.02														
26		0.09	0.04	0.21												
27				0.03												
28																
29																
30																
31																

Figure 15. One-Year Daily Hydrograph for Sub-Watershed No. 6

**TABLE 4. CALCULATED ONE YEAR STORM VOLUMES
FOR WILDE LAKE SUB-WATERSHEDS**

<u>Watershed</u>	<u>Storm Volume (Million Gallons)</u>	<u>Watershed</u>	<u>Storm Volume (Million Gallons)</u>
1	0.461	12	0.594
2	0.586	13	0.697
3	0.530	14	1.030
4	0.342	15	0.236
5	0.530	16	0.782
6	1.104	17	1.130
7	0.803	18	0.290
8	0.626	19	0.924
9	1.460	20	1.203
10	1.258	21	0.690
11	0.854	22	0.358

SECTION V

WATER QUALITY AND DEMANDS

Storm runoff has been stored and reused locally within the catchment area for centuries. Many island communities, such as Gibraltar, Guam, and Bermuda, have been entirely dependent on local efforts to maximize this type of water resource to supply their needs. The Department of Agriculture published handbooks to assist farmers in constructing farm cisterns as supplementary water supplies. In contrast to water falling on clean and well-maintained, frequently impervious catchments, the storm runoff from a typical urban area has acquired heavy pollutant loads before it enters a storm water collection system. In addition to discharging large quantities of pollutant into natural watercourses, storm runoff is sometimes permitted to enter combined sewer systems or may accidentally enter sanitary sewers through leaks, improper connections, etc., where it causes overflows and further increases stream pollution.

The reuse of urban runoff as a supplementary water resource presents a number of problems, detailed elsewhere in this report. It is clear that the treatment of storm water to drinking water quality is potentially a difficult and expensive task. Section VIII discusses the treatment problem in some detail. Since many possible water uses do not require drinking water quality, an investigation was made of applications of sub-potable water within an urban community. A literature search was made to discover other applications of storm water to sub-potable reuse. No local reuse of storm water was reported in the literature surveyed, but numerous applications of sub-potable water systems, including reclaimed wastewater, were reported (6, 7, 8, 10, 13, 20, 21, 22). Next, the kinds of reuse available in the Columbia area were investigated and listed. The various types of reuse were categorized into four water quality classes and criteria developed for estimating the total demand in each. Finally, demands were calculated for the Wilde Lake watershed and each of its component sub-watersheds.

A literature survey has been made of previously reported efforts to recover a wide variety of waste and sub-potable waters for distribution to uses that would otherwise be supplied by public potable water systems. This search was made to determine previous technical success with such projects but, more particularly, to determine levels of demand for various types of reuse and to discover whether public acceptance of such systems was achieved. The largest portion of reported applications was devoted to use of reclaimed wastewater, normally from the municipal wastewater treatment plant. Although this involves a water resource somewhat more difficult to treat and probably much more unlikely to win widespread acceptance, published experience has been summarized to permit an examination of a more severe case than the one under study.

WATER RECLAMATION

The expression "wastewater reclamation" is used to overcome public aesthetic - or, more properly, semantic - objections to the term "sewage treatment" (6). In a broad sense, "wastewater reclamation" is defined as the purposeful upgrading of the quality of sewage, rendering it reusable by agriculture, industry, or the public (11). Planned wastewater reclamation has been practiced for over 30 years. All the basic use categories except drinking water supply are being directly served by reclaimed waters. Even though the present quantity of reclaimed wastewater being used is still rather small, planned reclamation appears to be on the threshold of becoming a major consideration in the augmentation of water resources (9).

Wastewater reclamation falls into two categories: incidental and planned. Waste treatment plants discharging into fresh waters provide incidental reclamation in that the effluent may be used again. On the other hand, planned reclamation involves the production of water suitable for direct application to a beneficial use. Treatment facilities are financed and constructed specifically for this purpose (9).

Reclamation of wastewater may be directed toward any of the following applications:

- Specialized industrial uses

- Crop and domestic irrigation

- Recreational lakes and ponds

- Ground water recharge

- Non-potable domestic uses, e.g., toilet flushing (6)

A brief summary of reported experience in each of these areas of use follows.

INDUSTRIAL USES

Direct use of treated municipal wastewaters by industry is presently small, but the increasing number of undertakings attest to the enormous potential that exists (13). Most of the recirculation by industry has been limited to using industrial wastewater for cooling purposes and a few have virtually closed systems requiring only small amounts of makeup water (24). Mining, metallurgical, and allied industries located in arid regions of the United States have pioneered the reclamation of wastewater and many large industrial plants find it desirable to treat and reuse wastewater because of significant financial savings (10). The literature reports that more than 150 industries located in 38 states are reclaiming industrial wastewater (10). Two well-known examples are Bethlehem Steel

Company's plant at Sparrow's Point, Maryland, which uses effluent from Baltimore's domestic sewage treatment plant (13), and the Kaiser Steel Company at Fontana, California, which reclaims industrial and domestic wastes from its own plant (6).

The quantities of water used by industry may often be radically reduced by proper reuse. The steel industry offers the best example of this. At the Kaiser Steel Plant at Fontana, the net use of water has been reduced to 1000 gallons per ton of steel. This is a substantial saving when compared to the national average of 65,000 gallons per ton of steel. Not only will the reuse of water affect the total quantity of water used, but reused water can often be purchased at a considerably lower cost (23; see also 15-22).

AGRICULTURAL USES

Irrigation requires more water than any other use (24). Methods of irrigation with reclaimed water include: flooding, spraying or sprinkling, ridge and furrow, and subsurface irrigation. Although most state health standards do not permit the use of sewage for truck farming regardless of the degree of treatment, treated effluent can be used on cotton, beets, vegetables grown for seed production, pasture crops, and woodlands (13). Crop irrigation has been used as an aid to disposal of community wastes, with irrigation being merely incidental to the disposal operation. At Fresno, California, the city sewage farm takes the place of a secondary treatment plant that would be needed to sufficiently clean the wastewater for disposal to a canal or stream (10).

RECREATIONAL USES

One of the outstanding examples of recreational use of reclaimed wastewater is the installation at Golden Gate Park in San Francisco. This activated sludge plant is providing an effluent which is chlorinated and used for maintaining the level of decorative lakes and the irrigation of pastures, grass, and gardens. Irrigation of golf courses using reclaimed wastewater has increased in recent years. The Marine bases at El Toro and Pendleton, California, are examples. At El Cajon, near Palo Alto, California, there are semipublic courses now under construction which will use sewage effluent for irrigation. It has been shown that golf courses employing sewage effluent are able to maintain themselves without additional fertilization (10).

A reclamation project has been undertaken by the Santee County Water District in San Diego County, California, which provides for the development of a series of artificial lakes fed by the effluent of a standard activated sludge sewage treatment plant. There are four lakes, two of which are open to the public for recreational uses. Swimming is currently not permitted, and catching fish for human consumption is not allowed. However, a recreational area is provided, via use of reclaimed wastewater, in an area which affords little attraction of this type (13).

GROUND WATER RECHARGE

Replenishment of ground waters through artificial recharge has been given considerable attention. In many areas, ground water levels have been rapidly falling and are not being restored through natural means. Annual water replacement may lag behind withdrawals due to the slow movement rate of ground water and limited opportunities for surface waters to penetrate the earth's surface. Waters suitable for artificial recharge may be classified generally as flood waters, industrial wastes, and municipal sewage effluents (13).

Numerous methods are used in artificial recharge operations. The oldest and still most common technique is the utilization of holding basins. A second method is the use of a modified stream bed. A stream channel is widened and treated by a combination of methods to increase the percolation rate (13). Ditches and furrows may also be used. The basic arrangements are contour, lateral, and branching types. Where slopes are relatively uniform, flooding provides an economical means of recharge. Another recharge method is the pumping of water through injection wells directly into porous strata (13, 16).

The Whittier Narrows water reclamation project in Los Angeles County, California, is an example of ground water recharge potential (12). The object of this project is to conserve and assist in restoring the water resources of the agricultural area of Southern California. Treated sewage is distributed into spreading ponds. The reclaimed waters then percolate through to the ground water aquifers and are eventually used in the irrigation of agricultural lands. Operation of the Whittier Narrows plant is being closely watched by various agencies interested in public health, education, pollution control, and industry (13).

DOMESTIC USES

Domestic uses of reclaimed wastewater are generally limited to lawn watering and toilet flushing with a few notable exceptions. These exceptions are relatively recent developments. Some of the domestic uses reported are:

1. Sites of Grand Canyon National Park, where treated domestic wastewater was first used in the park in 1926 for toilet flushing, lawn sprinkling, cooling water, and boiler feedwater at the power plant.
2. Pomona, California, where municipal sewage plant effluent has been used for domestic irrigation of lawns and gardens in a suburban home development since 1929.
3. San Diego State Teachers College, California, where sewage effluent was first used for lawn and shrubbery irrigation in 1931.

4. Golden Gate Park, San Francisco, California, which was mentioned earlier, first used activated sludge plant effluent in 1932 for lawn watering and maintaining water levels in some of the park lakes (6).

A pilot plant in Tucson, Arizona, has demonstrated that potable water may be made from municipal waste treatment effluent. The process is automatic and uses effluent from an activated sludge process, followed by softening, flocculation, filtration, demineralization, and carbon absorption (14).

A more striking example is the experience of Chanute, Kansas. In 1956, when drought dried up the local water supply, effluent from the city's secondary treatment plant was diverted to the water supply reservoir. Treated wastewater was recycled through the city 8 to 15 times during a five-month period under the supervision of public health authorities (8, 14).

DUAL SYSTEMS

For situations where domestic water requirements can be partially met by a nearby source of reasonably clean water, a dual water system may be used. Dual water systems can be defined as those involving distribution of two grades of water to consumers through independent pipe networks (7).

The idea of dual water systems for domestic supply received consideration in the United States as early as 1894. At first, universal adoption of the concept was not advocated. It was urged, however:

"Where there is a supply of naturally pure water sufficient to meet culinary and drinking requirements, it (should) be supplied through a separate system, leaving the larger requirements to be furnished from the nearest, cheapest source of reasonably clean water" (7)

The water of Coalinga, a city of 6300 located in South Central California, is highly mineralized and unsatisfactory for drinking purposes and most home uses. Because of the poor quality of this supply, softer water is shipped to Coalinga from Armona, California, by railroad car. Since 1931, this water has been distributed to the homes through a system constructed in parallel with the hard water system. The hard water supply is disinfected for safety reasons. Thus, Coalinga has been operating a dual water system for over 35 years (7).

Avalon, a town on Catalina Island offshore from Los Angeles, operates a dual water system. This system has been in operation since 1914. A fresh water distribution system supplies water for drinking, cooking, and bathing. A salt water system supplies ocean water for fire protection and sanitary purposes. It appears that the operation of this system will continue due to the shortage of fresh water on the island (7).

Another example of a dual water system is the one in operation at the University of Wisconsin since sometime prior to 1913. The University draws water from Lake Mendota which is treated and pumped to the buildings and used for showers, laboratories, air conditioning, fire protection, boilers, and other uses not involving human consumption. Drinking water is purchased from the city of Madison and supplied to the same buildings through a separate distribution system (7).

For public health reasons, community-wide distribution of biologically safe and unsafe waters through a dual system cannot be seriously considered. If the question of dual water systems is examined on the condition that both supplies are safe for drinking purposes, then the concept appears worthy of study. The two supplies are usually termed "potable" and "sub-potable" (7).

The sub-potable supply should be maintained at a quality level such that occasional inadvertent use for drinking purposes will not result in harm to the consumer. The supply must be free from harmful biologic forms and toxic chemicals. Elements of the sub-potable system that furnishes water for normal and fire fighting demands must provide hydraulic capacity for fire flows plus the coincident draft for toilet flushing, irrigation, and other domestic and industrial uses.

PROBLEMS AFFECTING PUBLIC ACCEPTANCE

The literature is essentially silent on public reaction to the aesthetics of wastewater reclamation and dual water systems. It can be observed, however, that the public has accepted existing projects. Even though this acceptance has occurred, the aesthetics of reclamation are still considered an important factor (9).

Hundreds of U.S. cities draw their water from inland streams. They are in fact, using the diluted sewage effluent of upstream cities. Although the presence of such pollution is generally known, consumer acceptance of the water does not appear to be appreciably reduced. Dilution and time appear to dissipate aesthetic objections. An assumed public aversion to the concept of direct wastewater reclamation has been a major deterrent to expansion of this practice (9).

When dual water systems are mentioned, objections regarding health hazards are raised. These objections arise from associating the word "dual" with the hazardous combination of potable and contaminated water (7). Early in the development of dual water systems, it was established that the most serious difficulty in operating such systems "will be found in the proper regulation of the use of purified water" (7). It was felt that some customers would use the pure water for other than intended uses, while others, either carelessly or to avoid cost, would not use it at all.

Conversely, the two water supplies in existing dual water systems are both safe for drinking purposes. The "potable" supply is maintained at an excellent quality, suitable in all respects for drinking, cooking, and bathing. The "sub-potable" supply, inferior in chemical quality but equal in safety, is suitable for waste transport, lawn and garden irrigation, street flushing, fire protection, and other uses not requiring water of high quality. Each dwelling served by a dual water system requires two service lines and two meters (7).

The difference between treatment systems for water reuse and wastewater treatment for disposal to streams seems to be one of degree rather than of kind. At least no sharp distinction is evident in the literature. Streams and lakes are natural systems for bio-treatment. Their water is often reused several times before discharge to the sea. Present public acceptance of "natural" processes is considered greater than acceptance of engineering processes (4).

REUSE CLASSIFICATIONS

Storm water was considered as a potential water source for every level of water use existing within the Wilde Lake area. Although each discrete use has its own unique water quality requirement or standard, an estimate will be made to draw the various uses into a limited number of categories, simplifying further analysis. Whenever a group of use types is defined as one category, the water quality standard selected for that category must meet the requirements of each use considered.

Water use can be broadly described as being either residential, commercial, industrial, or public. Table 5 lists various types of residential water uses, with approximate percentages of total consumption. Linaweaver et al. collected data from more than 3000 residences in public metered water and public sewer areas of the eastern United States and analyzed that data in terms of residential consumption not including sprinkling use and sprinkling use alone (26). All areas studied in the eastern U.S. averaged 56.7 gallons per capita per day for nonsprinkling residential use. The thirteen local areas studied ranged from 42.9 to 71.1 gallons per capita per day (27). Examination of similarities between areas studied by Linaweaver and the Wilde Lake region of Columbia led to the selection of 61.5 gallons per capita per day as an estimate of water consumption within the dwelling unit.

Sprinkling use was conclusively correlated, in the Linaweaver work, to the irrigable area associated with the dwelling unit and the local deficiency between potential evapotranspiration and summer precipitation (28). Three areas were located within the immediate vicinity of Columbia for which data were available, known as Glenmont, English Manor, and Northwest Branch Estates. All are served by the Washington Suburban Sanitary Commission and are comprised of homes similar in value and lot size to those in Columbia. Average annual sprinkling use, as computed by Linaweaver, divided by average irrigable area for each of these locations

resulted in sprinkling usages ranging from 253 gallons per day per acre to 921 gallons per day per acre (27). In order to arrive at a value for estimating demand in Columbia, these three areas were averaged, and the mean was found to be 518 gallons per day per acre. Table 5 was revised, with slight regrouping of uses, due to certain uses sharing common plumbing fixtures, making separation unfeasible. The revised table is shown as Table 6.

TABLE 5. DISTRIBUTION OF RESIDENTIAL
WATER USE, AFTER REID (25)

	<u>% of Total</u>	<u>Gal/Capita/Day</u>
Toilet flushing	28	24.0
Bathing	23	20.0
Dishwashing	4	3.75
Drinking and other kitchen	3	2.75
Laundrying	10	8.5
Lawn sprinkling	29	25.0
Auto washing	3	2.5

TABLE 6. DISTRIBUTION OF RESIDENTIAL
WATER USE - REVISED

<u>Quality</u>	<u>Use</u>	<u>Gal/ Capita/Day</u>	<u>Gal/ Day/Acre</u>
AA	Human consumption, food preparation, general kitchen use	6.5	
A	Bathing, laundrying, auto washing	31.0	
B	Lawn sprinkling		518.0
C	Toilet flushing	24.0	

The water quality considerations shown in Table 6 have been assigned in decreasing order to quality level. Water quality Class "AA" is defined as meeting the U.S. Public Health Service Drinking Water Standards and is intended for all potable water uses (29). Class "A" is virtually identical to Class "AA" except for taste and odor considerations not important in water which will not actually be consumed or used in food preparation. Class "B" has somewhat more lenient requirements, particularly with respect to suspended solids, except that it must be disinfected to the

same standard as Class "AA," thus providing safety for accidental ingestion. Class "C," also disinfected, has little more than a suspended solids limit and minimal requirements for corrosivity. Specific limits on various quality parameters are shown in Table 7.

Commercial water use might contain all of the use categories discussed above in addition to air conditioning loads and other miscellaneous commercial uses. In all cases, the water demands of each commercial user must be evaluated with respect to the four water quality levels described above and defined in Table 7. Particular types of commercial use vary widely not only in the specific uses involved, but in the relative magnitude of each of them.

Industrial uses fall, to a great degree, into the same type of breakdown as commercial uses. Additionally, industry frequently has large demands for process water, whose required quality varies from very low to substantially higher than potable water quality, depending on the industry and the process employed. In order to compute demands for each water quality, a particular industry must be analyzed in detail to determine its requirements by quality level. In the study area, the Wilde Lake watershed of Columbia, no process-water-using industry is contemplated, making this type of analysis unnecessary in the present study.

Public water uses include applications such as street cleaning, sewer flushing, fire protection, and snow melting. Street cleaning and sewer flushing account for very little water use on a long-term basis, compared to the available supply. Providing water for fire protection implies that high flows will be available from a system of very high reliability. Fire insurance rating bureaus normally insist that the required fire flows for a community like Columbia be available entirely from elevated storage, independent of pumping capacity. The lowest requirement in the Wilde Lake watershed applies to the detached single family residential areas, where 120,000 gallons of storage must be continuously available. In order to consider a number of separate systems, each of them must have the full fire flow capability. Furthermore, the area under study is already served by the Howard County Metropolitan Commission with a conventional public water supply system designed to provide necessary fire flows. For these reasons, fire flows were not considered as possible uses of the storm water within the local reuse concept. Another public use considered was the flushing of streets following snowfalls to melt and remove accumulated snow. This is nonconsumptive use from the standpoint of the storm water system, since the runoff returns to the storage facility. This type of snow removal is confined to those occasions when pavement temperature is above 40°F and air temperature is at least 30°F, to prevent refreezing of the water as ice. The flushing must also be done at a time when the pavement will have the opportunity of drying prior to the next freezing cycle, effectively preventing use of the technique at any time except under bright sunlight. Unfortunately, the circumstances which permit the use of water flushing of snow usually result in rapid melting of the snow without flushing. The water required on the infrequent occasions when this procedure is indicated is considered to be a negligible amount.

**TABLE 7. MAXIMUM CONCENTRATION OF
SELECTED POLLUTANTS BY REUSE CATEGORY**

Constituent (mg/l unless indicated)	Required Water Quality* Maximum Concentration			
	AA	A	B	C
Alkyl benzene sulfonate	0.5	0.5	0.5	0.5
Ammonia (NH ₃)	0.5	0.5	0.5	0.5
Arsenic	0.01	0.05	0.05	0.05
Barium	1.0	1.0	1.0	1.0
Cadmium	0.01	0.01	0.01	0.01
Calcium	75	75	75	75
Carbon chloroform extract	0.2	0.2	0.2	0.2
Chloride	250	250	250	250
Chromium (hexavalent)	0.05	0.05	0.05	0.05
Copper	1.0	1.0	1.5	1.5
Cyanide	0.01	0.2	0.2	0.2
Fluoride	1.5	3.0	3.0	3.0
Iron	0.3	0.3	0.3	0.3
Lead	0.05	0.1	0.1	0.1
Magnesium	50	150	150	150
Magnesium+Sodium sulfate	500	500	1000	1000
Manganese	0.05	0.1	0.5	0.5
Nitrate (as NO ₃)	45	50	50	50
Oxygen, dissolved (minimum)	5.0	5.0	4.0	4.0
Phenolic compounds (as phenols)	0.001	0.002	0.002	0.002
Selenium	0.01	0.05	0.05	0.05
Silver	0.05	0.05	0.05	0.05
Sulfate	250	250	400	400
Total solids	500	500	500	500
Zinc	5.0	15	15	15
BOD (5-day) mg/l	---	---	---	---
Coliform (MPN per 100 m l)	1	70	240	240
pH (units)	7.0	6.0	6.0	6.0
Color (units)	15	20	30	30
Turbidity (units)	0-3	3-8	8-15	15-20
Suspended solids	---	---	10**	30**
Phosphates	1.0	1.0	1.0	1.0
Volatile suspended solids	---	---	---	---

*Based upon maximum concentrations allowed by the USPHS, the World Health Organization, and the Water Quality Standards of the State of Maryland.

**Higher suspended solids are permitted by various water quality standards. Limit based on sediment control and water contact recreation.

DISTRIBUTION OF DEMANDS

In order to permit evaluation of various levels of treatment for each of the sub-watersheds and combinations of sub-watersheds, the water demands were tabulated, by water quality class, for each sub-watershed. The boundaries of the sub-watersheds were revised slightly for this tabulation in order that the area of computation be similar to that which would actually be serviced by a discrete distribution system within the sub-watershed. Figure 16 shows the distribution areas, divided by broken lines, except for the sub-watersheds lying wholly outside the Columbia project, where the natural drainage lines will continue to define the areas. A second tabulation was made of land uses within the revised sub-watersheds, analogous to that shown previously as Figure 14. This tabulation, shown as Figure 17, also lists irrigable area, defined as the difference between total area and impervious area, excepting open space. This is the approximate area of all lawns, gardens, and school grounds which are likely to be irrigated by lawn sprinklers during the summer.

The usage coefficients described earlier for each of the water quality classes are applied to the appropriate parameters and residential water use is computed for each quality level. Other demands, including commercial and institutional uses, are estimated individually for each installation and listed on the sheet. Residential and other demands can be totalled to yield water demand by water quality class for the sub-watershed. This operation was performed on each sub-watershed in the entire drainage area to evaluate total demands. Prior to assembling this information for input to the systems analysis computer program, however, the demands listed for Class "B" uses, lawn sprinkling, were derated. The storage requirements associated with several patterns of water demand were evaluated. It was found that the nonuniform behavior of sprinkling demands would place a significant requirement on reservoir size. Since other sources of supply would be required during certain periods, an analysis was performed to determine the extent to which sprinkling demands could be met with reservoirs of reasonable size.

It was determined that when sprinkling demands were satisfied with an 87.5 percent reliability and the balance of the demand with a 100 percent reliability, the reservoir requirement was essentially independent of the fraction of total demand which was composed of sprinkling uses. In other words, sprinkling demands are interchangeable with other demands, provided no more than 87.5 percent of the sprinkling use is satisfied by the system. The balance of the demands would be supplied from the public water system through a makeup water connection. As a result of this analysis, sprinkling demands in the watershed were reduced to 87.5 percent of their original value, then treated on the same basis as other, uniform demands with respect to storage requirements. The methods used in making this analysis are described in Section VII.

Examination of the water use categories indicates that eight possible combinations of water use exist. This includes all possible combinations of the lower three quality classes, as well as all four classes together.

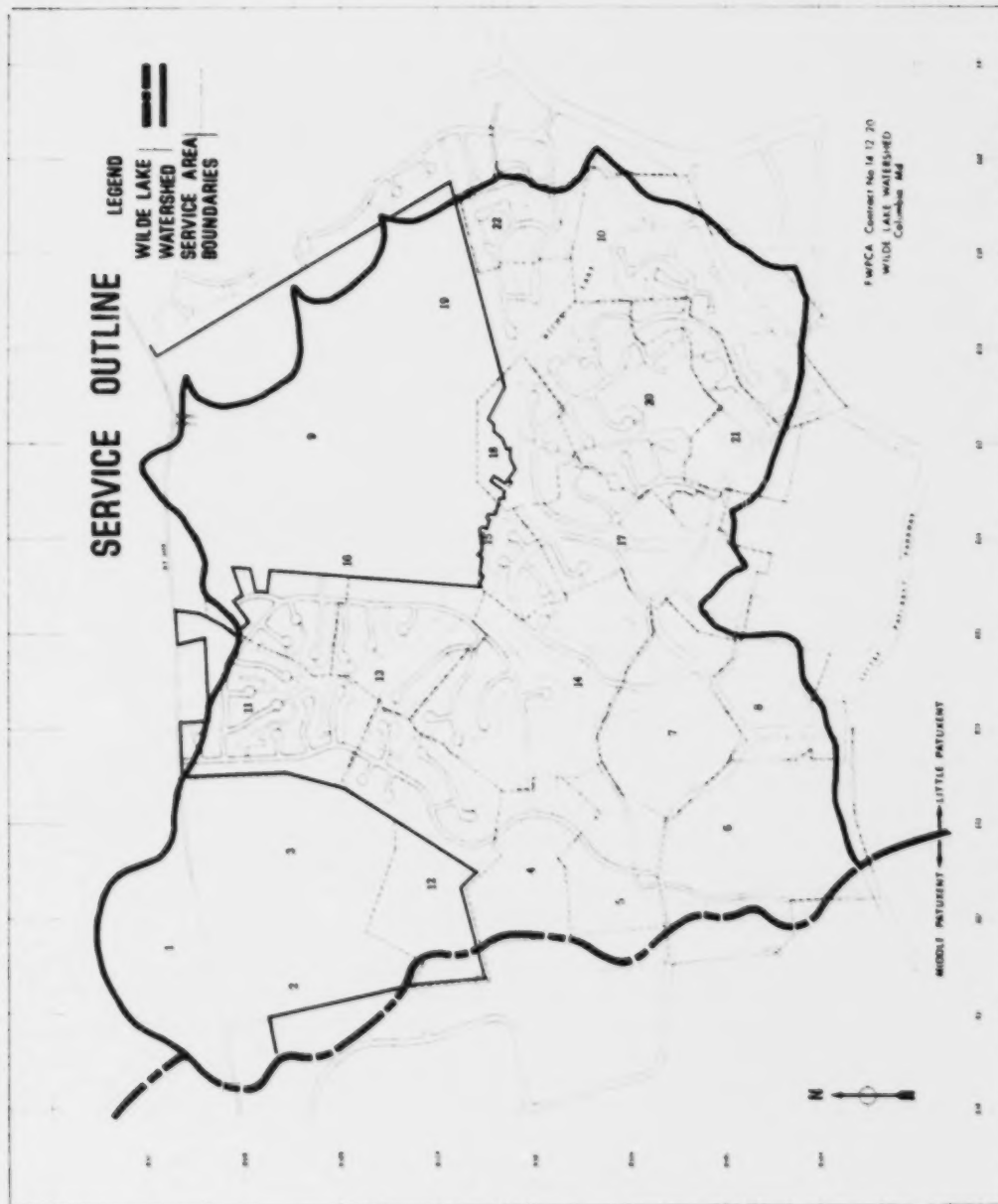


Figure 16. Wilde Lake Watershed - Service Areas

	LAND USE	AREA (Ac.)	UNITS/ Ac.	RESID. UNITS	POPUL./ UNIT	POPUL.	% IMPERV.	IMPERV. AREA (Ac.)
COLUMBIA PROJECT	OPEN SPACE	10.7					0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Single Family - Medium Density	12.5	4.0	50	4.0	200	16.0	2.0
	Town-Houses	4.9	11.0	54	3.5	189	33.0	1.6
	Garden Apartments	6.8	17.0	116	2.5	290	20.0	1.4
	Mid-Rise Apartments	3.6	28.0	101	2.5	253	22.0	0.8
	Employment Centers	14.					100.0	14.0
	School Sites						18.0	
	Public Rights-of-way	9.5					85.0	8.1
	OPEN SPACE						0.0	
NON- COLUMBIA	Single Family - Low Density		1.2		4.0		7.0	
	Public Rights-of-way						85.0	
	COLUMBIA TOTAL	620		321		932	45.0	27.9
	NON-COLUMBIA TOTAL							
	AREA TOTAL	620		321		932	45.0	27.9

ESTIMATED WATER DEMANDS (A/VG. DAY)						DEMAND (G/D)
RESIDENTIAL DEMANDS {	IRRIGABLE AREA	POPULATION X 6.5	G/D/Person	(AA)	=	6,060
		POPULATION X 31.0	G/D/Person	(A)	=	28,890
		= 23.4 Ac. X 518.0	G/D/Acre	(B)	=	12,120
		POPULATION X 24.0	G/D/Person	(C)	=	22,370
OTHER DEMANDS {	ESTIMATED DEMAND OF WILDE					(AA) = 3,800
	LAKE VILLAGE GREEN SHOP-					(A) = 400
	PING CENTER, LIBRARY, AND COMMUNITY CENTER.					(C) = 8,750

Figure 17. Work Sheet - Typical Sub-Watershed Land Use
Tabulation and Water Demand Calculation

It was assumed that, should potable water be produced, there would be no incentive for separating plumbing systems, therefore all demands would be connected to a single system. In all the other cases, where sub-potable water is being produced, the plumbing systems constructed for sub-potable water distribution might be connected to any combination of fixtures.

The water demands computed for each sub-watershed, after derating of the Class "B" demand as discussed above, were combined into each of the possible use combinations and displayed on a table. This table was input to the computer program to serve as the basis for evaluating various alternate reuse systems. A printout of this table appears as Figure 18.

**** OUTPUT ****

**** WATER DEMANDS BY SERVICE LEVEL ****

WATERSHED	AA	A	B	C	A,B	A,C	A,B,C	B,C	AA,A,B,C
1	550.0	2600.0	7210.0	2020.0	9810.0	4620.0	11830.0	9230.0	12380.0
2	550.0	2600.0	7298.0	2020.0	9898.0	4620.0	11918.0	9318.0	12468.0
3	210.0	990.0	3120.0	770.0	4110.0	1760.0	4880.0	3890.0	5090.0
4	1960.0	9180.0	6799.0	7480.0	15979.0	16660.0	23459.0	14279.0	25419.0
5	2550.0	12150.0	4078.0	9410.0	16228.0	21560.0	25638.0	13488.0	28188.0
6	17310.0	82550.0	20983.0	63910.0	103533.0	146460.0	167443.0	84893.0	184753.0
7	9080.0	43310.0	11830.0	33530.0	55140.0	76840.0	88670.0	45360.0	92750.0
8	4050.0	18230.0	7980.0	15060.0	26210.0	33290.0	41270.0	23040.0	45320.0
9	2730.0	13000.0	36930.0	10100.0	49930.0	23100.0	60030.0	47030.0	62760.0
10	45460.0	33670.0	9520.0	34760.0	43190.0	68430.0	77950.0	44280.0	123410.0
11	3150.0	15000.0	12100.0	11620.0	27100.0	26620.0	38720.0	23720.0	41870.0
12	1480.0	7070.0	5800.0	5470.0	12870.0	12540.0	18340.0	11270.0	19820.0
13	2340.0	11160.0	10330.0	8640.0	21490.0	19800.0	30130.0	18970.0	32470.0
14	5280.0	23500.0	15910.0	21690.0	39410.0	45190.0	61100.0	37600.0	66380.0
15	810.0	3840.0	3130.0	2980.0	6970.0	6820.0	9950.0	6110.0	10760.0
16	2310.0	11040.0	21480.0	8540.0	32520.0	19580.0	41060.0	30020.0	43370.0
17	6060.0	28890.0	10610.0	22370.0	39500.0	51260.0	61870.0	32980.0	67930.0
18	1200.0	5700.0	5710.0	4420.0	11410.0	10120.0	15830.0	10130.0	17030.0
19	1140.0	5460.0	12190.0	4220.0	17650.0	9680.0	21870.0	16410.0	23010.0
20	3460.0	17800.0	17590.0	16280.0	35390.0	34080.0	51670.0	33870.0	55130.0
21	5430.0	25890.0	10560.0	20040.0	36450.0	45930.0	56490.0	30600.0	61920.0
22	1460.0	6940.0	6210.0	5380.0	13150.0	12320.0	18530.0	11590.0	19990.0

Figure 18. Summary of Water Demands
(gallons per day)

SECTION VI

STORM WATER QUALITY

In order to specify the various treatment alternatives which might be considered in the development of the optimum system to achieve the desired water quality requirement, it is necessary that the pollutional content of the storm water be known to some reasonable degree. At the least, the nature and approximate maximum levels of concentration of the principal pollutants should be known. The first attempt towards determining these parameters was to search published reports for analysis of urban runoff from similar areas. Numerous references to storm water quality were located, but only a few went further than isolated, unrelated pieces of data (30-38). Among the several extensive studies of storm water pollution, most failed to relate specific pollutants to conditions in the watershed such as land use, industry, industrial and commercial wastes, etc. Pollutant concentrations quoted seldom included any reference to the intensity of runoff at the time of sampling or the hydrograph previous to the sampling time. One of the more complete studies of storm water pollution was performed in Cincinnati, Ohio, by Weibel et al., yet it did little more than set up some broad ranges of pollutant concentrations for watersheds similar to the one analyzed (31). Table 8 summarizes the observed range of a number of pollutants found in storm water at a variety of locations throughout the world. Each of the studies used to develop this table were sufficiently comprehensive to exclude the possibility of a chance or "freak" occurrence of an unrepresentative concentration of some specific pollutant. It can be seen that many of the items vary by two or more orders of magnitude throughout reported experience. It was concluded that the literature survey would be of little value in establishing a pollutant baseline for the Columbia location, due to the very wide variance of reported data. The sections below describe the tentative listing of maximum concentrations for expected pollutants, followed by the results of the water quality sampling station, intended to verify the tentative data.

TENTATIVE DESCRIPTION OF RUNOFF QUALITY

A review of the specific pollutants mentioned in various studies led to a classification of the parameters as shown in Table 9.

Solids

None of the studies reported above contained any quantitative data on floating solids. Inlet structures are normally expected to exclude large floating objects (boards, tree branches, toys, etc.) but the existence of open drainage channels downstream from the closed storm water collection system permits such objects to enter the flow. In addition, leaves, small branches, paper or cardboard objects, etc., are commonly found in all storm water collection systems. Much of the debris will eventually absorb water and sink, joining suspended solids, but some will remain bouyant until it is manually or mechanically removed from the stream.

TABLE 8. REPORTED STORM WATER
POLLUTANT CONCENTRATIONS*

	<u>Observed Range</u>	<u>Number Locations</u>
Dissolved Solids	30 - 8,000	1
	154 - 228	1
Suspended Solids	26 - 36,250	6
Volatile	38 - 98	1
Nonvolatile	119 - 292	1
BOD	6.9 - 625	9
COD	18 - 3,100	2
DO	6.4 - 8.0	1
Total Nitrogen (NO ₂ , NO ₃ , Organic)	2.3 - 11.8	3
Total Phosphate	0.47 - 1,400	2
Coliform Total/100 ml	40 - 240,000	4
Fecal Strep/100 ml	median- 20,500	
Chlorides	11 - 160	2

*Refs. 30, 31, 34, 35, 36, 37, 38

TABLE 9. PRINCIPAL INDICES OF
STORM WATER POLLUTION

<u>Type</u>	<u>Pollutant</u>	<u>Unit of Measure</u>
Solids	Floating Solids	lb/day
	Suspended Solids	mg/l
	Dissolved Solids	mg/l
Oxygen Balance	Biochemical Oxygen Demand (BOD)	mg/l (5-day)
	Chemical Oxygen Demand (COD)	mg/l
	Dissolved Oxygen (DO)	mg/l
Nutrients	Total Nitrogen (N, NO ₂ , NO ₃ , etc.)	mg/l
	Total Phosphorous (P, PO ₄ , etc.)	mg/l
Bacteria	Coliform Group	MPN/100 ml
	Fecal Streptococci	MPN/100 ml
Miscellaneous	pH	pH units
Physical and	Temperature	°C
Chemical	Oil and Grease	mg/l
Parameters	Chlorides	mg/l
	Pesticides	mg/ml
	Surfactants	mg/l

To a degree, the nature and size of these objects will be determined by the physical layout of the collection system. No estimate was made for the incidence of floating solids.

Suspended solids include all those particles so fine or so bouyant that they are suspended in the stream. It may include both particles which will settle following loss of stream velocity and those which are either colloidal or at neutral bouyancy. These solids may or may not be volatile in nature. The volatile solids are generally composed of various organic substances, both animal and vegetable in origin, which are characterized by putrescibility and eventual decomposition into simpler, stable materials. Volatile solids of vegetable origin may include cut and fallen vegetation (leaves, grass, etc.), humus, and discarded food. Animals contribute offal and carcasses of various sizes and stages of decomposition. Non-volatile suspended solids are primarily soils and general surface debris and are inherently stable and inorganic in nature. Analysis of runoff from similar watersheds in the Cincinnati study cited earlier led to the tentative assumption that, after construction is completed, runoff from Columbia will probably not exceed 300 mg/l suspended solids, 60 mg/l of which might be volatile.

Dissolved solids may include various soluble minerals and salts, as well as small quantities of other organic and inorganic materials which might be slightly soluble. Decomposition of volatile suspended solids often produces soluble organic products. Some dissolved materials have been precipitated from the atmosphere by the rainfall and were dissolved before striking the ground, but most have entered the water as suspended solids and have subsequently dissolved. Many dissolved substances are of interest themselves as specific pollutants, such as chlorides, pesticides, surfactants, etc. Since these separately discussed pollutants are the only dissolved solids expected to be of any interest in the Columbia runoff, no limit was created for dissolved solids.

Oxygen Balance

Biochemical Oxygen Demand (BOD) is an index determined by a laboratory procedure for measuring the amount of oxygen consumed, in five days, by natural agencies in stabilizing the volatile material in the water. It is a reasonable estimate of the oxygen that will be consumed in nature as the various organic materials decompose. Untreated domestic sewage has a BOD on the order of 200 mg/l, while Maryland Department of Water Resources water quality criteria classify as "good" a stream having less than 3.5 mg/l BOD. It was estimated that the storm runoff might contain up to 25.0 mg/l BOD.

Chemical Oxygen Demand (COD) measures the amount of oxygen consumed under specified conditions in the oxidation of organic and oxidizable inorganic matter, corrected for the influence of chlorides. COD and BOD tend to complement one another, although there is no inherent relation between the two. The difference is primarily one of the mechanism of oxidation. In the present analysis, the BOD test is considered more appropriate than COD, so no estimate was made of storm water COD.

Dissolved Oxygen (DO) measures the quantity of oxygen available in the water in molecular form. It may range from 0.0 up to saturation, which is a function of water temperature and, to a lesser extent, atmospheric pressure. Saturation levels may range, at sea level, from 7.1 mg/l at 35°C to 14.4 mg/l at 1°C. It can be assumed that falling raindrops are 100 percent saturated and that any loss of oxygen content is a result of rising temperature or of physical contact with oxidizable contaminants, or both. It was assumed that the DO of storm water might be as low as 0.0 mg/l.

Nutrients

Both nitrogen and phosphorous are effective nutrients and are commonly found in water as nitrates and phosphates. Nitrogen might also be present as nitrites and as dissolved or organic nitrogen, due to the decomposition of organic material. The presence of either of these materials is usually indicative of decomposing organic materials in the stream or in the watershed, or of the direct application of chemical fertilizers containing these materials to land in the watershed, resulting in a high nutrient level of runoff. The presence of these substances in excessive concentrations contributes to algal blooms and to distortion of the ecological balance of the receiving body of water. It was assumed that the runoff under study would not exceed 3.0 mg/l. Nitrogen (measured as NO_3) or 1.0 mg/l phosphorous (measured as PO_4).

Bacteria

Due to the great difficulty of isolating and identifying all of the possible pathogenic (disease-producing) bacteria which might be present in storm water, it is common practice to test for presence of one of the fecal bacteria and use that strain as an indicator of fecal contamination. If such contamination is found to be present, it is considered possible that pathogens might also be present. If no fecal bacteria are identified, the presence of pathogens is considered unlikely. The fecal bacteria used most widely as an indicator is the coliform group of bacteria, particularly the *Escherichia Coli*, a relatively large rod-like bacteria. It is readily detected by means of any of several standard laboratory methods. Since coliform bacteria may be present in soil as well as humans and animals, more specific indicators, such as the fecal streptococci group, are sometimes used. In all cases, test results are reported as most probable number (MPN) per 100 ml. Since public health considerations dictate that chlorination should be practiced regardless of the bacterial concentrations, no limit was suggested for the probable bacterial pollution.

Miscellaneous Physical and Chemical Parameters

From the many parameters that could be considered under this heading, the only ones which are expected to be of any significance in the Wilde Lake area are chlorides, oil, and grease. No quantitative estimate was made for oil and grease, but it is virtually certain that runoff from streets, driveways, and parking lots will include noticeable quantities. It is estimated that chlorides, primarily from salt used to melt snow in the

winter, may range as high as 160 mg/l. Table 10 lists the tentative storm water quality description for the Wilde Lake watershed.

TABLE 10. EXPECTED MAXIMUM POLLUTANT CONCENTRATIONS FOR STORM WATER IN WILDE LAKE WATERSHED

Type	Pollutant	Expected Max. Concentration
Solids	Floating Solids	Present
	Suspended Solids	300 mg/l
	Volatile Suspended Solids	60 mg/l
Oxygen Balance	BOD	25 mg/l
Nutrients	Nitrogen (N, NO ₂ , NO ₃) (as NO ₃)	3.0 mg/l
	Phosphorous (P, PO ₄) (as PO ₄)	1.0 mg/l
Miscellaneous Physical and Chemical Parameters	Chlorides	160 mg/l
	Oil and Grease	Present

The results of the water quality sampling program conducted in the Wilde Lake watershed are contained in Appendix B.

WATER QUALITY STANDARDS FOR WILDE LAKE

The Maryland Department of Water Resources has recently adopted a comprehensive set of water quality standards regulating the state's interstate and principal intrastate waters, in compliance with federal requirements (40). Although Wilde Lake is not specifically assigned a minimum water quality in these standards, its intended use is clearly consistent with the Class "C" uses outlined in the regulations - water contact recreation, fish propagation, agricultural and industrial water supply. These standards make the following requirements of bodies of water intended for Class "C" uses:

1. Fecal coliform bacteria less than 240 MPN/100 ml
2. Dissolved oxygen greater than or equal to 4.0 mg/l at any time, 5.0 mg/l monthly average
3. pH greater than or equal to 6.0, less than or equal to 8.5
4. Temperature less than or equal to 93°F (35°C)

In addition, general requirements are made, prohibiting harmful concentrations of any other pollutants which might be present. Harmful concentrations are to be determined in terms of the intended uses of the body of water which is to be protected.

A conservative interpretation of these water quality standards and the ability of Wilde Lake to neutralize pollutants entering it from storm water runoff led to the definition of effluent standards for all water deliberately discharged into Wilde Lake or its tributary watercourses. These standards take into account the possibility of spills during severe storms and have been further adjusted to coincide with the water quality criteria adopted for Class "C" water uses in the previous section. Table 11 lists the parameters being used to define effluent water quality together with the maximum concentration of each.

TABLE 11. EFFLUENT STANDARDS FOR
WATER DISCHARGED INTO WILDE LAKE*

Solids	Floating Solids	None
	Suspended Solids	30.0 mg/l
	Volatile Suspended Solids	None
Oxygen Balance	BOD	3.5 mg/l(DO ₂ BOD)
Nutrients	Nitrogen (as NO ₃)	50.0 mg/l
	Phosphorous (as PO ₄)	1.0 mg/l
Bacteria	Fecal Coliform	240 MPN/100 ml
Miscellaneous Physical and Chemical Parameters	pH	6.0 or greater
	Oil and Grease	None
	Chlorides	250 mg/l
	Surfactants (ABS)	0.5 mg/l
	Ammonia (NH ₃)	0.5 mg/l
	Arsenic	0.05 mg/l
	Barium	1.0 mg/l
	Cadmium	0.01 mg/l
	Calcium	200 mg/l
	Carbon Chloroform Extract	0.2 mg/l
	Chromium (hexavalent)	0.05 mg/l
	Copper	1.5 mg/l
	Cyanide	0.2 mg/l
	Fluoride	3.0 mg/l
	Iron	0.3 mg/l
	Lead	0.1 mg/l
	Magnesium	150 mg/l
	Manganese	0.5 mg/l
	Phenolic Compounds (as phenols)	0.002 mg/l
	Selenium	0.05 mg/l
	Silver	0.05 mg/l
	Sulfate	400 mg/l
	Zinc	15 mg/l
	Color	30 color units

*Refs. 40 and 41

SECTION VII

STORM WATER STORAGE

The Wilde Lake watershed of Columbia is similar to other recent suburban developments in its approach to the storm drainage problem. The natural drainage channels have been retained with their original topography and the floodplains are used, in part, to satisfy the open space requirement of the community planning concept. The floodplains are normally surrounded with building lots and these, in turn, are circumscribed by roads. The storm drain system consists of many relatively small inlet and pipe systems, each draining independently into a floodplain. After discharge, the storm water follows natural drainage lines to Wilde Lake. Roads are generally located at or near the ridge lines of the various sub-drainage areas and only a few cases exist within the Wilde Lake watershed where an appreciable amount of runoff is collected by a single closed collection system before discharge to a floodplain.

CONSTRUCTION TYPES

The early planning of the Local Reuse System study provided for consideration of unconventional storage facilities, such as natural or artificial underground cavities, slip-formed vaults beneath streets, etc., as well as more conventional approaches to storage within the floodplain. Examination of the drainage area indicated that significant quantities of storm water were not available at any place in the watershed, except at locations within the natural floodplains. Of the 22 sub-watersheds considered, only number 10 enters Wilde Lake as a wholly closed system without having been substantially collected within a natural floodplain. When storage is to be provided in the floodplain, two types were considered - open ponds and constructed storage. In sub-watersheds where floodplain space is not available, such as number 10, conventional constructed storage was considered, along with unconventional approaches, such as special vaults under roads and parking lots. Construction costs for such vaults were found to be on the order of \$0.20/gallon of storage capacity and maintenance problems were considered to be severe. Since conventional constructed storage is competitive with other methods from sizes of 50,000 gallons upward and maintenance is facilitated by the improved access, parametric costs for unconventional approaches were not developed and no further consideration of them was judged necessary.

In the case of constructed storage, costs are dependent on location with respect to existing grade. In most cases, the storage facility must be placed below grade to permit gravity inflow under all conditions. Occasionally, local topography may permit a tank to be constructed partially or completely above grade. To maintain maximum flexibility, the below grade and above grade cases were separately analyzed and costed. Tanks of steel and of prestressed concrete construction were also investigated. Open ponds were studied and the optimum combination of excavated and

natural storage determined for ponds in topography similar to Columbia. Table 12 summarizes the types of reservoir construction which were studied and cost estimated.

**TABLE 12. STORM WATER STORAGE
RESERVOIR CONSTRUCTION TYPES**

Natural Storage	Open Ponds
Constructed Storage	Steel Tanks - Above Grade
	Steel Tanks - Below Grade
	Concrete Tanks - Above Grade
	Concrete Tanks - Below Grade

SIZING CRITERIA

The storm water storage facilities included in the Local Storage, Treatment, and Reuse of Storm Water concept function as multiple purpose reservoirs and, as such, are subject to several potentially conflicting requirements. Their basic purposes, in the order of the priority implied by the orientation of this study, are water quality control, water supply, and the creation of a pleasing and harmonious asset to the local environment. As water quality control reservoirs, they are expected to be designed and operated so as to prevent degradation of their contents, maintain sufficient release of the natural watercourse to protect downstream uses, and possibly effect sedimentation. The water supply use dictates certain relationships of storage capacity, supply, and yield, dependent on, among other things, the reliability with which the yield must be maintained. Finally, the appearance of the facility must be at least consistent with its surroundings and might well be designed to be an asset to the area. The presence of the facilities must be acceptable to the community and pose no particular safety hazard or nuisance.

In studying size requirements for the local storage units, the individual reservoir locations were first analyzed to determine the yield versus capacity relationships, assuming 100 percent reliability over the five-year period of the synthetic hydrograph. Similar analyses were performed to determine relationships with 99, 98, 95, 90, and 80 percent reliability for the five-year period. These analyses were performed for constant yields and for yields approximating the patterns of several different levels of lawn sprinkling use. Next, water quality considerations were analyzed in terms of operating and design requirements, and the storage units studied as sedimentation basins. Finally, sizes and design criteria were developed which met requirements imposed by all three purposes. The following sections describe in detail the development of these criteria.

STORAGE/YIELD CHARACTERISTICS FOR WATER SUPPLY USE

A common approach to water supply reservoir sizing problems is the Mass Diagram or Rippl Method (42). When performed graphically, this method plots cumulative supply against cumulative yield so that cumulative surplus or deficiency can be examined. A simple graphical manipulation is sufficient to measure maximum required storage directly for the time period plotted. The identical result may be obtained with somewhat better resolution by applying the following continuity equations to tabular data:

$$Q_i + S_i = D_i + S_{i+1} \quad (19)$$

where:

$$\begin{aligned} Q_i &= \text{inflow on the } i^{\text{th}} \text{ day} \\ S_i &= \text{storage volume at the start of the } i^{\text{th}} \text{ day} \\ D_i &= \text{yield on the } i^{\text{th}} \text{ day} \end{aligned}$$

This equation may be applied to each successive time period throughout the available record, and the maximum value found for S_i is the maximum required storage, comparable to that found by the graphical technique. In order to achieve meaningful results in small watersheds subject to large, short-term fluctuations in runoff rates, a five-year daily synthetic hydrograph was developed (Section IV), containing 1825 data points. These data were developed for each of the 22 sub-watersheds. Each of these must be analyzed over a wide range of demand levels, or yields. This last requirement is made to permit an economic tradeoff between the revenue of additional water uses and the incremental cost of additional storage. A preliminary appraisal of the difficulty of a conventional mass balance continuity analysis of 1825 data points for each of the watershed yield combinations led to investigation of alternate approaches to the reservoir sizing problem.

The selected method was that of Residual Mass Tabulation, a technique for approximating the result of the method described above with many fewer calculations and in a manner much better suited to the high speed processing characteristics of the digital computer (42). Table 13 is a portion of a residual mass tabulation for one of the sub-watersheds in the Wilde Lake area. The two left-hand columns list total accumulated runoff (in cfs-days) for various periods (in days) in the synthetic record. The first line indicates runoff for the driest day of record; the second line indicates the total runoff for the driest five consecutive days of record; the third line indicates the total for the driest 10 consecutive days of record; etc. For example, the eleventh line indicates that during the driest 60 consecutive days of record, the total accumulated runoff was 1.0 cfs-day. This listing continues until the period listed equals the total record, 1825 days, and the accumulated runoff equals the total runoff for the period, 83.09 cfs-day in this example. The second

TABLE 13. EXAMPLE OF RESIDUAL MASS TABULATION -
SUB-WATERSHED NO. 8

Period (days)	Runoff (cfs-days)	Demand(cfs)=0.0125			Demand(cfs)=0.0200			Demand(cfs)=0.0400			Demand(cfs)=0.0600		
		Outflow (cfs-days)	Storage (cfs-days)		Outflow (cfs-days)	Storage (cfs-days)		Outflow (cfs-days)	Storage (cfs-days)		Outflow (cfs-days)	Storage (cfs-days)	
1	0.0	0.0125	0.0125		0.0200	0.0200		0.0400	0.0400		0.0600	0.0600	
5	0.0	0.0625	0.0625		0.1000	0.1000		0.2000	0.2000		0.3000	0.3000	
10	0.0	0.1250	0.1250		0.2000	0.2000		0.4000	0.4000		0.6000	0.6000	
15	0.0	0.1875	0.1875		0.3000	0.3000		0.6000	0.6000		0.9000	0.9000	
20	0.0	0.2500	0.2500		0.4000	0.4000		0.8000	0.8000		1.2000	1.2000	
25	0.0	0.3125	0.3125		0.5000	0.5000		1.0000	1.0000		1.5000	1.5000	
30	0.0	0.3750	0.3750		0.6000	0.6000		1.2000	1.2000		1.8000	1.8000	
35	0.12	0.4375	0.3175		0.7000	0.5800		1.4000	1.2800		2.1000	1.9800	
40	0.22	0.5000	0.2800		0.8000	0.5800		1.6000	1.3800		2.4000	2.1800	
50	0.37	0.6250	0.2550		1.0000	0.6300		2.0000	1.6300		3.0000	2.6300	
60	1.00	0.7500	-0.2500		1.2000	0.2000		2.4000	1.4000		3.6000	2.6000	
80	1.54	1.0000	-0.5400		1.6000	0.0600		3.2000	1.6600		4.8000	3.2600	
100	1.70	1.2500	-0.4500		2.0000	0.3000		4.0000	2.3000		6.0000	4.3000	
150	3.32	1.8750	-1.4450		3.0000	-0.3200		6.0000	2.6800		9.0000	5.6800	
200	4.65	2.5000	-2.1500		4.0000	-0.6500		8.0000	3.3500		12.0000	7.3500	
250	7.53	3.1250	-4.4050		5.0000	-2.5300		10.0000	2.4700		15.0000	7.4700	
300	9.97	3.7500	-6.2200		6.0000	-3.9700		12.0000	2.0300		18.0000	8.0300	
350	12.33	4.3750	-7.9550		7.0000	-5.3300		14.0000	1.6700		21.0000	8.6700	
400	13.35	5.0000	-8.8500		8.0000	-5.8500		16.0000	2.1500		24.0000	10.1500	
450	15.58	5.6250	-9.9550		9.0000	-6.5800		18.0000	2.4200		27.0000	11.4200	
500	10.00	6.2500	-11.7500		10.0000	-8.0000		20.0000	2.0000		30.0000	12.0000	
600	23.05	7.5000	-15.5500		12.0000	-11.0500		24.0000	0.9500		36.0000	12.9500	
700	29.52	8.7500	-20.7700		14.0000	-15.5200		28.0000	-1.5200		42.0000	12.4800	
800	33.71	10.0000	-23.7100		16.0000	-17.7100		32.0000	-1.7100		48.0000	14.2900	
900	39.03	11.2500	-27.7800		18.0000	-21.0300		36.0000	-3.0300		54.0000	14.9700	
1000	43.37	12.5000	-30.8700		20.0000	-23.3700		40.0000	-3.3700		60.0000	16.6300	
1200	52.26	15.0000	-37.2600		24.0000	-28.2600		48.0000	-4.2600		72.0000	19.7400	
1400	61.56	17.5000	-44.0600		28.0000	-33.5600		56.0000	-5.5600		84.0000	22.4400	
1600	72.27	20.0000	-52.2700		32.0000	-40.2700		64.0000	-8.2700		96.0000	23.7300	
1825	83.09	22.8120	-60.2780		36.5000	-46.5900		73.0000	-10.0900		109.5000	26.4100	
Maximum	Storage		0.3750		0.6300			3.3500			(supply deficit)		

pair of columns, headed "DEMAND (CFS) = 0.0125," show the method of analysis for a constant yield of 0.0125 cfs. The column headed "OUTFLOW" lists the total yield for each of the periods listed in the first column (number of days times daily yield). The column headed "STORAGE" lists the volume drawn from storage found by subtracting the "RUNOFF" column from the "OUTFLOW" column. The largest positive number in this column is the approximate size, in cfs-day, of the required reservoir, given this demand. This process is repeated successively with various demands until the entire possible operating range has been explored. In each case, the largest positive number in the storage column is taken as the required reservoir size. When the "STORAGE" column does not show a negative entry in the last row, a supply deficit is indicated. That is, the total available supply is less than the total required yield and no finite reservoir can be constructed which would provide this yield during the period analyzed.

After the residual mass tabulation has been performed for a sufficient number of yields to describe the range of possible operation, a plot is made of yield versus storage requirement. A typical plot is shown on Figure 19. In this case, the units of storage have been converted from cfs-day to gallons and those of yield from cfs to gallons/day. Since the method used samples of various time periods and computes storage for the sampled periods only, the storage requirement found will be only an approximation of the actual requirement. It may be equal to the actual requirement or, more probably, slightly smaller. For this reason, the plot shows a smooth curve intersecting or passing above the plotted points. This curve can be taken as a continuous relationship between storage requirements and yield for the period of the synthetic hydrograph used to generate the points. Figure 20 shows the storage/yield characteristic of another sub-watershed as well as the results of a graphical Rippl Method solution for the same area. The correspondence between the two methods was considered satisfactory for the purposes of this study, and all subsequent storage analyses were made by the method of residual mass tabulation, using digital computers.

STORAGE/YIELD CHARACTERISTICS FOR SPRINKLING USES

The determinations of storage requirements described above assumed that the demand on the storage facility, or yield, would be constant in terms of volume/day. This assumption is relatively valid for domestic uses such as toilet flushing, bathing, laundering, food preparation, etc. The only significant use which cannot be treated as a constant demand is lawn sprinkling. In addition to its totally seasonal characteristic in many areas of the country, the pattern of use is highly irregular from day to day. In their major study on residential water use, Linaweaver and others found the relationships between the most severe and the average periods of sprinkling use in a residential area near the Columbia site (27). These are shown in Table 14.

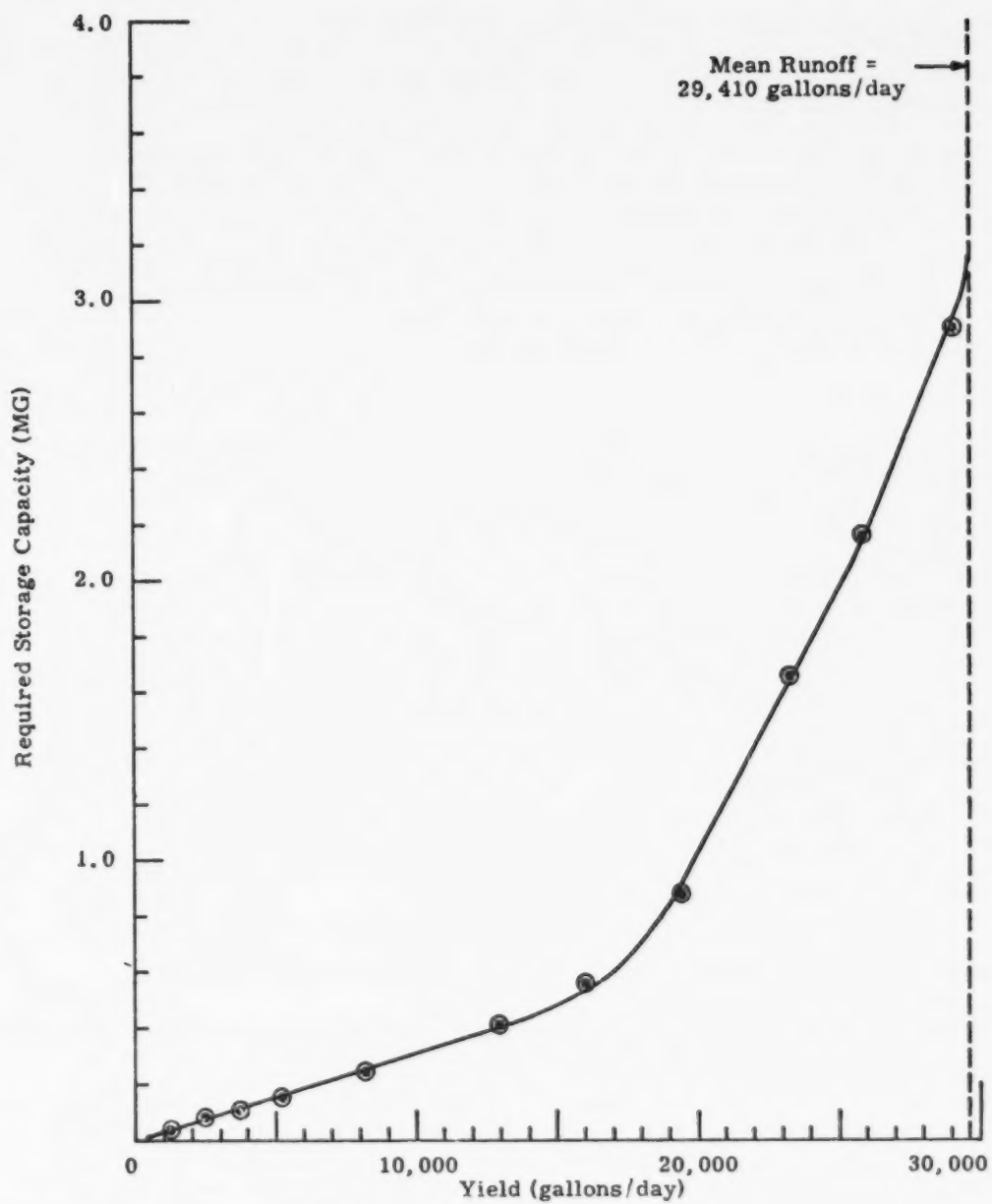


Figure 19. Storage vs. Yield
Example of Residual Mass Tabulation Analysis
Sub-Watershed Number 8

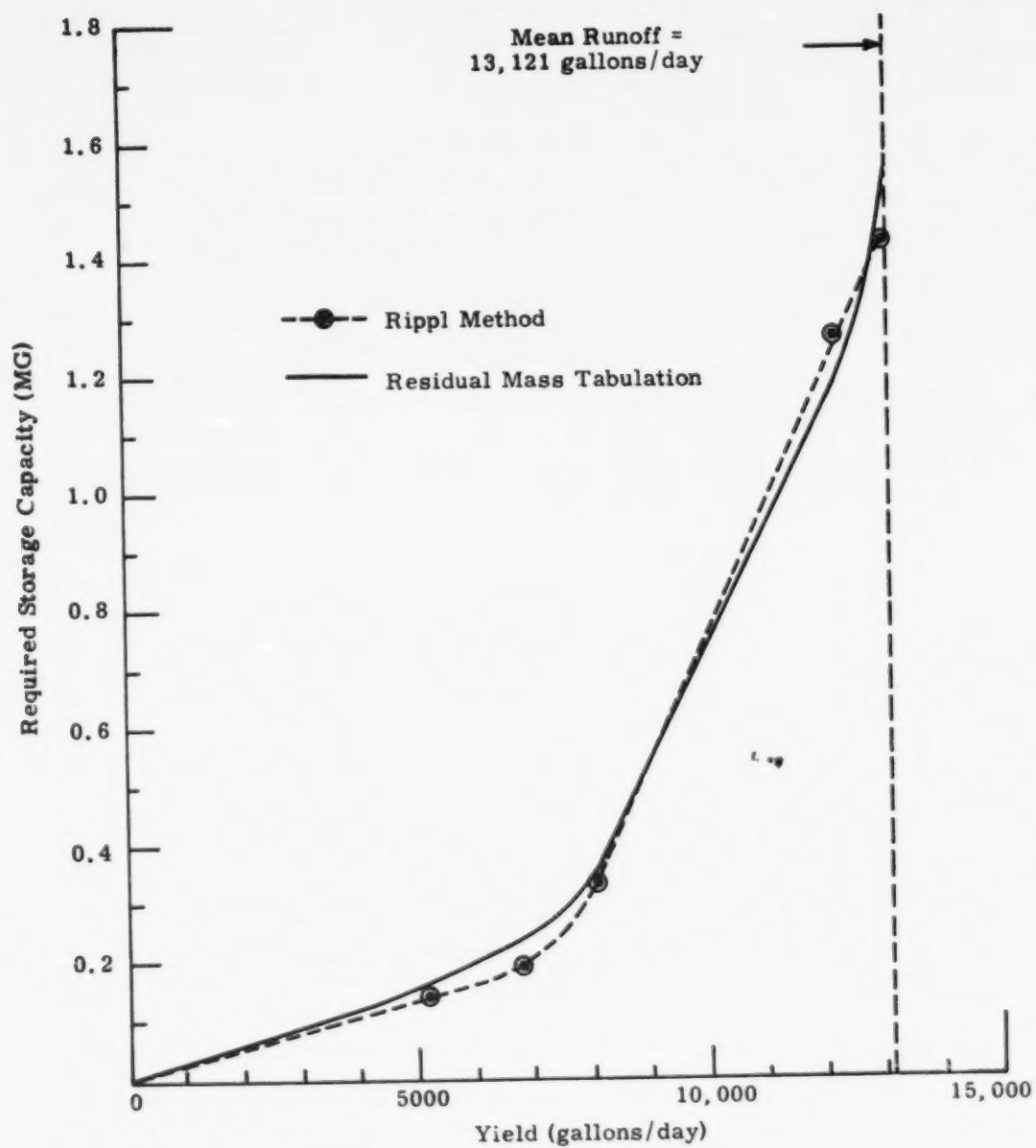


Figure 20. Storage vs. Yield -
Comparison of Rippl Method and Residual Mass Tabulation
Sub-Watershed Number 4

TABLE 14. RATIO OF SPRINKLING USE DURING SELECTED PERIOD TO AVERAGE SPRINKLING USE

Glenmont and Northwest Branch Estates,
Washington Suburban Sanitary Commission,
October 1963 to September 1965

<u>Time Period</u>	<u>Ratio of Sprinkling Use to Average Use (27, p. 6)</u>
Peak Hour	21.20
Maximum Day	7.17
Maximum 92 Days	2.16
365 Days	1.00

A continuous curve was fitted to the four data points listed by a parabolic regression technique, yielding the following equation:

$$R_i = 7.26 d^{-0.336*} \quad \text{* Standard error of estimate = 0.0061; } \sigma = 0.993$$

where:

- R = ratio between average sprinkling use during period, d, and annual average sprinkling use
- d = period of sprinkling use being studied in days

This equation can now be used to estimate the ratio of sprinkling use during the most severe period of a selected length to the average sprinkling use. When average sprinkling use levels are developed in Section V and applied to this expression, the most severe sprinkling use can be evaluated for any period. Since the average use levels and ratios employed have been selected from studies of similar residential areas in the same geographical and climatological area, only the irrigable area is required as an input to this calculation.

This approach to evaluation of sprinkling demand patterns can be applied to the residual mass tabulation method of reservoir design to determine the reservoir requirements for a demand that is to be entirely or partially dedicated to sprinkling uses. The only additional assumption required is that the most severe sprinkling demands will coincide with the driest periods of record. Although dry periods may occur during seasons when evapotranspiration is very low and no sprinkling would be practiced, the assumption is clearly a conservative one and can be observed to conform to experience in a high percentage of actual cases. Table 15 shows the modified tabulation for a sub-watershed with parallel analysis of a 15,000 gallon/day average yield directed to uses which are zero percent

TABLE 15. EXAMPLE OF MODIFIED RESIDUAL MASS TABULATION -
STORAGE VS. PROPORTION OF DEMAND USED FOR SPRINKLING LAWNS

Sub-watershed #16, Demand (cfs) = 0.0232
(g/d) = 15,000

Period (days)	Runoff (cfs-days)	0% Sprinkling		Ratio	25% Sprinkling		Ratio	100% Sprinkling		Storage
		Outflow	Storage		Outflow	Storage		Outflow	Storage	
1	0.0	0.0232	0.0232	2.565	0.0595	0.0595	7.260	0.1684	0.1684	0.1684
5	0.0	0.1160	0.1160	1.807	0.2096	0.2096	4.229	0.4906	0.4906	0.4906
10	0.0	0.2320	0.2320	1.588	0.3634	0.3634	3.351	0.7774	0.7774	0.7774
15	0.0	0.3480	0.3480	1.481	0.5154	0.5154	2.924	1.0176	1.0176	1.0176
20	0.0	0.4640	0.4640	1.414	0.6561	0.6561	2.654	1.2315	1.2315	1.2315
25	0.0	0.5800	0.5800	1.366	0.7923	0.7923	2.463	1.4285	1.4285	1.4285
30	0.0	0.6960	0.6960	1.329	0.9250	0.9250	2.316	1.6119	1.6119	1.6119
35	0.10	0.8120	0.7120	1.300	1.0556	0.9556	2.200	1.7864	1.6864	1.6864
40	0.18	0.9280	0.7480	1.276	1.1841	1.0041	2.103	1.9516	1.7716	1.7716
50	0.30	1.1600	0.8600	1.238	1.4361	1.1361	1.951	2.2632	1.9632	1.9632
60	0.86	1.3920	0.5320	1.209	1.6329	0.8229	1.835	2.5543	1.6943	1.6943
80	1.30	1.8560	0.5560	1.167	2.1660	0.8660	1.666	3.0921	1.7921	1.7921
100	1.46	2.3200	0.8600	1.137	2.6378	1.1778	1.546	3.5867	2.1267	2.1267
150	2.89	3.4800	0.5900	1.087	3.7828	0.8928	1.349	4.6945	1.8045	1.8045
200	4.05	4.6100	0.5900	1.056	4.8998	0.8498	1.224	5.6794	1.6294	1.6294
250	6.70	5.8000	-0.9000	1.034	5.9972	-0.7028	1.136	6.5888	-0.1112	-0.1112
300	9.12	6.9600	-2.1600	1.017	7.0783	-2.0417	1.069	7.4402	-1.6798	-1.6798
350	11.45	8.1200	-3.3300	1.004	8.1525	-3.2975	1.015	8.2418	-3.2082	-3.2082
Maximum Storage			0.8600			1.1778			2.1267	

sprinkling, 25 percent sprinkling, and 100 percent sprinkling. The "RATIO" columns are determined by evaluating Equation 20 for each time period and adjusting this ratio as follows:

$$R_{i1} = p R_i + (1-p) \quad (21)$$

where:

$$\begin{aligned} R_{i1} &= \text{adjusted ratio for } i^{\text{th}} \text{ period} \\ R_i &= \text{ratio from Equation 20 for } i^{\text{th}} \text{ period} \\ p &= \text{fraction of use assumed as sprinkling} \end{aligned}$$

These adjusted ratios are then entered in the "RATIO" columns in the rows corresponding to the correct time period. The tabulation proceeds as before, except that the "DEMAND" column is the product of the average daily demand, the number of days in the period, and the corresponding adjusted ratio. The results of this tabulation indicates significant increase in required storage as a direct result of an increasing proportion of the water being used to irrigate lawns. Figure 21 presents the results of a complete tabulation for one sub-watershed showing a family of curves for various proportions of sprinkling use. Figure 22 is another presentation of the same data, comparing sprinkling use fraction to required storage for a range of total average daily use levels. As discussed in Section V, this type of analysis was used to derive the derating factors for sprinkling uses used in the study.

STORAGE /YIELD CHARACTERISTICS AS A FUNCTION OF RELIABILITY

Both the constant use and the sprinkling use storage characteristics investigated above have been oriented toward a requirement of 100 percent reliable operation of the storage system based on a five-year synthetic hydrograph. Since the Local Storage, Treatment, and Reuse of Storm Water concept required that the system function only as a supplemental supply to a conventional public water distribution system, the storage requirements associated with various levels of system reliability were also analyzed. For purposes of this analysis, 90 percent reliability is defined as the ability of the system to supply at least 90 percent of the total water demand in any given year. Once again, the residual mass tabulation was employed to determine storage/yield characteristics. In this case, an assumption was made that the portion of the total demand which the storage system would fail to supply would be that occurring during the driest period. Table 16 is an example of a tabulation of a sub-watershed with a 15,000 gallon/day total demand which would be either 100 percent, 98 percent, 95 percent, or 90 percent supplied by the reservoir under study. In each case, the "DEMAND" column is reduced by the annual difference between reservoir yield and demand for each year or fraction thereof. In the example given, the

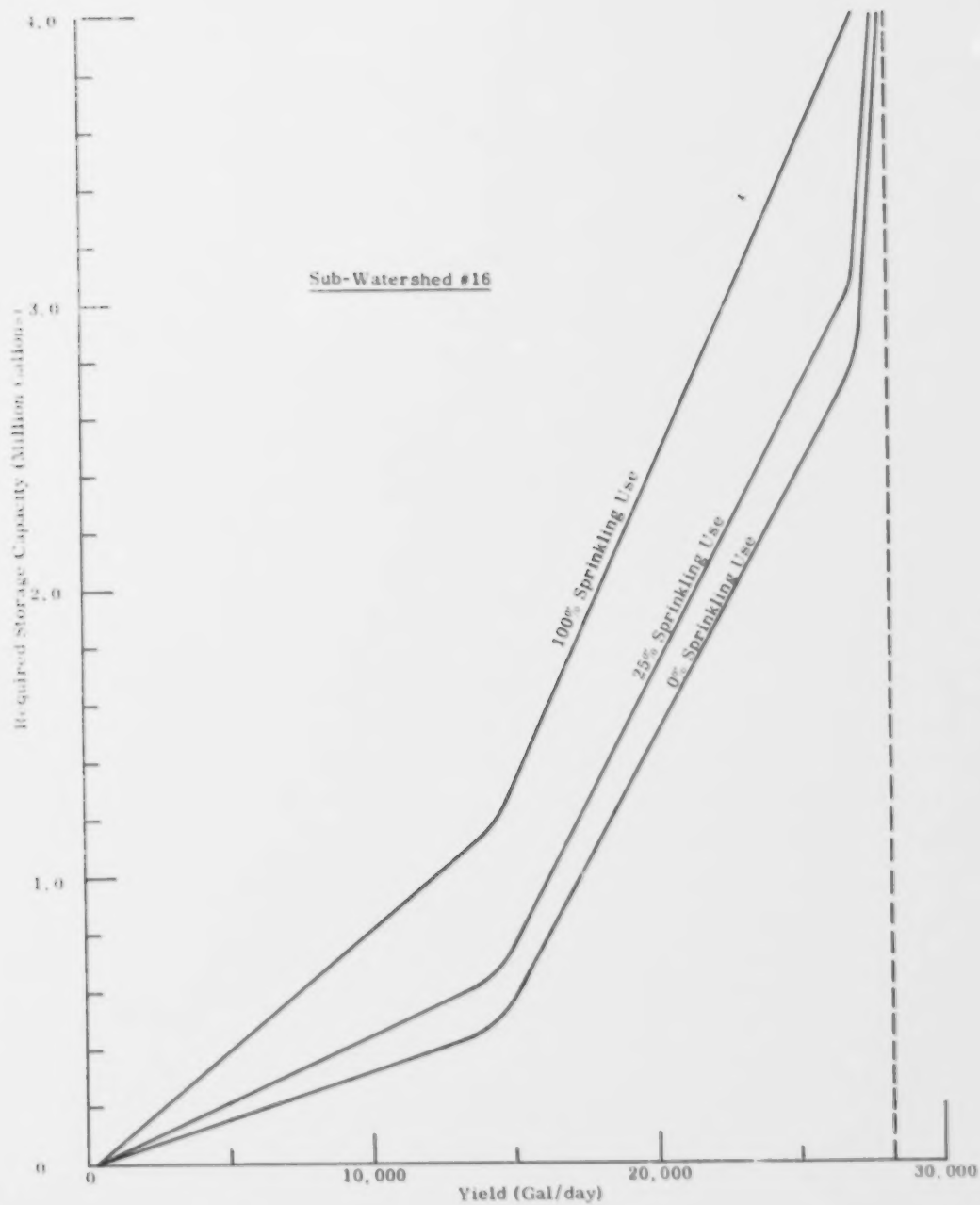


Figure 21. Storage Yield Characteristic vs. Proportion of Demand Used for Sprinkling Lawns

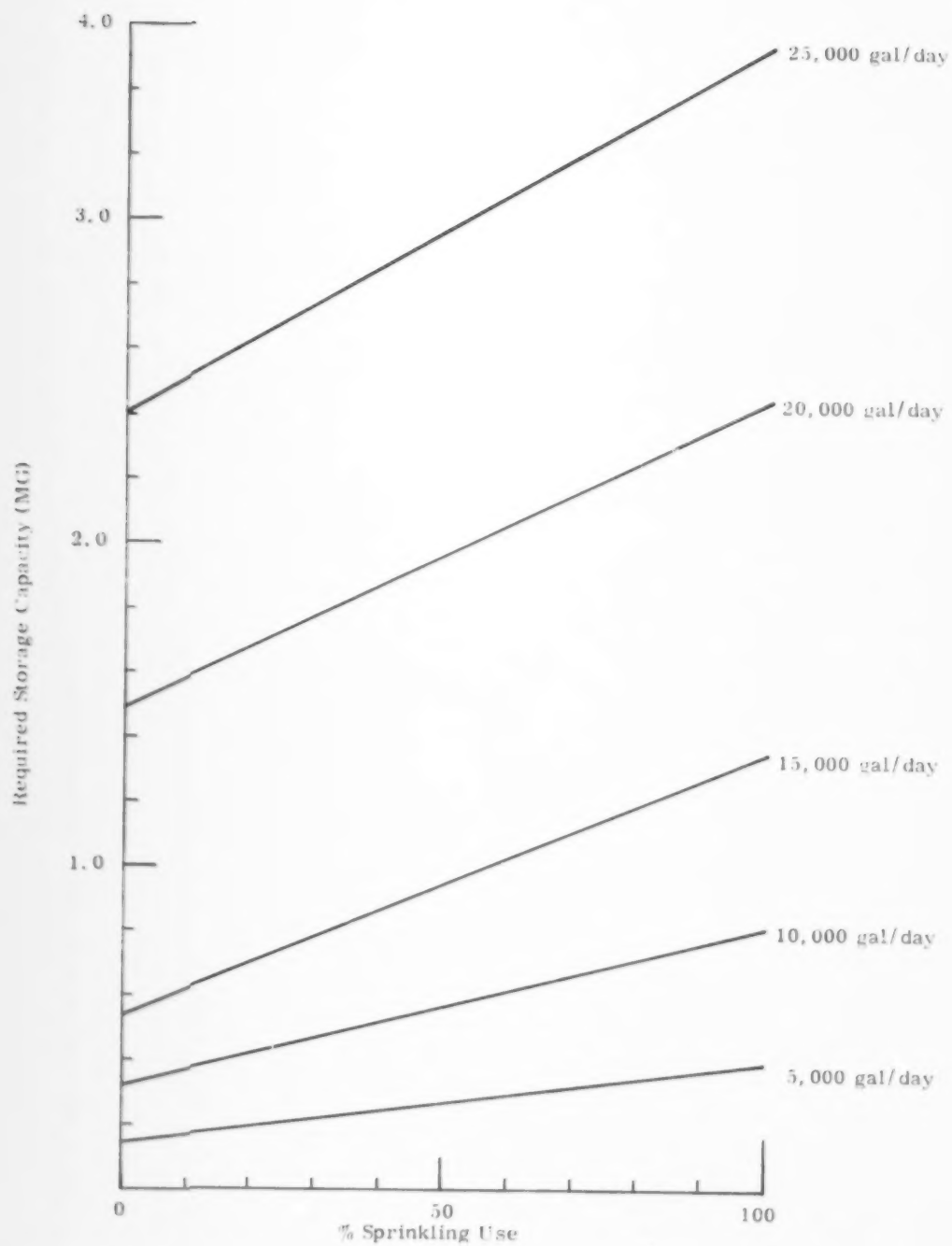


Figure 22. Storage/Sprinkling Use Characteristic vs. Yield, Sub-Watershed Number 16

TABLE 16. EXAMPLE OF MODIFIED RESIDUAL MASS TABULATION -
STORAGE VS. RELIABILITY

Sub-watershed #16, Demand (cfs) = 0.0232
(g/d) = 15,000

Period (days)	Runoff (cfs-days)	100% Reliability		98% Reliability		95% Reliability		90% Reliability	
		Outflow	Storage	Outflow	Storage	Outflow	Storage	Outflow	Storage
1	0.0	0.0232	0.0232	0.0	----	0.0	----	0.0	----
5	0.0	0.1160	0.1160	0.0	----	0.0	----	0.0	----
10	0.0	0.2320	0.2320	0.0626	0.0626	0.0	----	0.0	----
15	0.0	0.3480	0.3480	0.1786	0.1786	0.0	----	0.0	----
20	0.0	0.4640	0.4640	0.2946	0.2946	0.0405	0.0405	0.0	----
25	0.0	0.5800	0.5800	0.4106	0.4106	0.1565	0.1565	0.0	----
30	0.0	0.6960	0.6960	0.5266	0.5266	0.2725	0.2725	0.0	----
35	0.10	0.8120	0.7120	0.6426	0.5426	0.3885	0.2885	0.0	----
40	0.18	0.9280	0.7480	0.7586	0.5786	0.5045	0.3245	0.0810	-0.0990
50	0.30	1.1600	0.8600	0.9906	0.6906	0.7365	0.4365	0.3130	0.0130
60	0.86	1.3920	0.5320	1.2226	0.3626	0.9685	0.1085	0.5450	-0.3150
80	1.30	1.8560	0.5560	1.6866	0.3866	1.4325	0.1325	1.0090	-0.2910
100	1.46	2.3200	0.8600	2.1506	0.6906	1.8965	0.4365	1.4730	0.0130
150	2.89	3.4800	0.5900	3.3106	0.4206	3.0565	0.1665	2.6330	-0.2370
200	4.05	4.6400	0.5900	4.4706	0.4206	4.2165	0.1665	3.7930	-0.2570
250	6.70	5.8000	-0.9000	5.6306	-1.0694	5.3765	-1.3235	4.9530	-1.7470
300	9.12	6.9600	-2.1600	6.7906	-2.3294	6.5365	-2.5835	6.1130	-3.0070
350	11.45	8.1200	-3.3300	7.9506	-3.4994	7.6965	-3.7535	7.2730	-4.1770
Maximum Storage			0.8600		0.6906		0.4365		0.0130

98 percent column is reduced by 0.1694 cfs-day* for all time periods of one year or less, by 2×0.1694 cfs-day for time periods of two years or less but more than one year, etc. A complete analysis of the sub-watershed for various levels of reliability is shown in Figure 23. The same information is displayed in Figure 24 as the required storage as a function of supply reliability for various levels of demand.

The previous paragraphs have summarized an investigation of storage requirements for the 22 sub-watersheds in the Wilde Lake area, derived as a continuous function of such variables as total demand, proportion of demand devoted to lawn sprinkling, and supply reliability. This analysis has indicated that storage requirements are highly nonlinear with demand, relatively sensitive to sprinkling uses, and highly sensitive to the availability of supplemental water from other sources. The method of analysis employed, a modified residual mass tabulation technique, was well suited to the application of digital computer methods and produced satisfactory results within the scope of the initial assumptions. Since the same simulated rainfall record was applied to all sub-watersheds, little error will be introduced by summing the cumulative runoff columns of the tabulations for several sub-watersheds to analyze the characteristics of the combination. Unfortunately, it is necessary to perform the complete tabulation for each possible combination of sub-watersheds in order to determine the storage/yield characteristic for each possible reservoir application.

WATER QUALITY CRITERIA

In providing for detention of water for any purpose, reasonable care must be taken to prevent degradation of the water in storage. In the application under study, possibilities exist for degradation as a result of decomposition of suspended or floating organic materials, flushing of previously settled solids into downstream flows, growth of algae in the case of open storage facilities, and the creation of nuisance conditions due to breeding of flies and mosquitoes from aquatic larvae. Fortunately, most organic materials found in storm water are not readily putrescible and may be retained for a reasonable period before appreciable decomposition occurs. Periodic removal of accumulated materials is normally sufficient to avoid difficulty from this source, as well as minimizing the possibility of flushing settled material downstream. Design criteria can be developed to minimize spills, further reducing the occurrence of this problem. Algal blooms can be controlled by chemical treatment and by the maintenance of a relatively high throughput as a proportion of average contents, which will serve to minimize the growth of insect larvae as well. Provision must be made for sufficient attention to the condition of the reservoir, such as daily inspections, to preclude the formation of a quality degradation condition.

It would be desirable to exclude trash from the storage facility by the installation of a mechanical trash screen at the storm water inlet. Such an installation, however, would be prone to accumulate trash and clog during periods of high flow, thus causing overflows and possible flood damage.

* $0.1694 \text{ cfs-day} = [(100 - 98)15,000/646,316 \times 100] / 365$

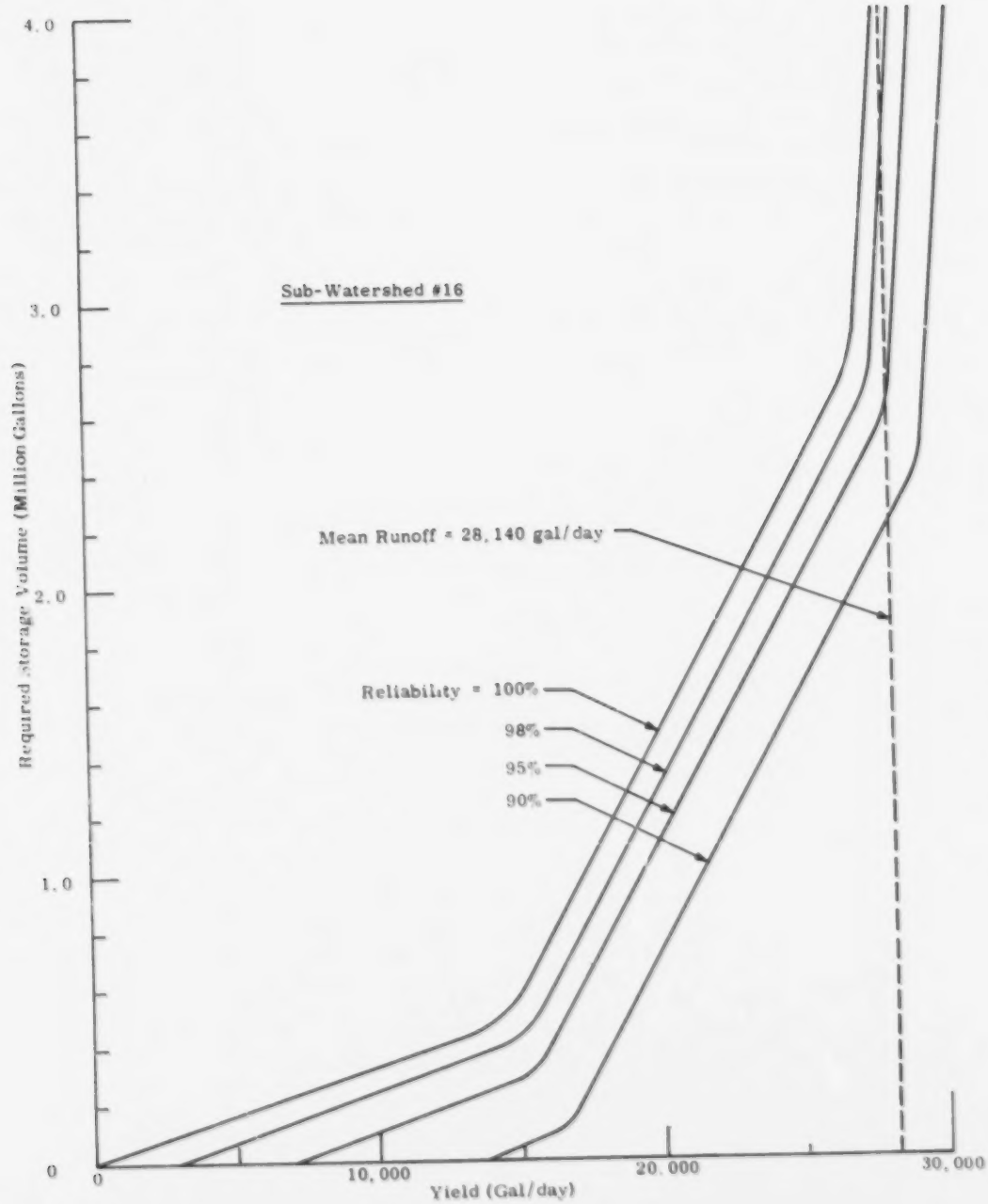


Figure 23. Storage/Yield Characteristic vs. Reliability

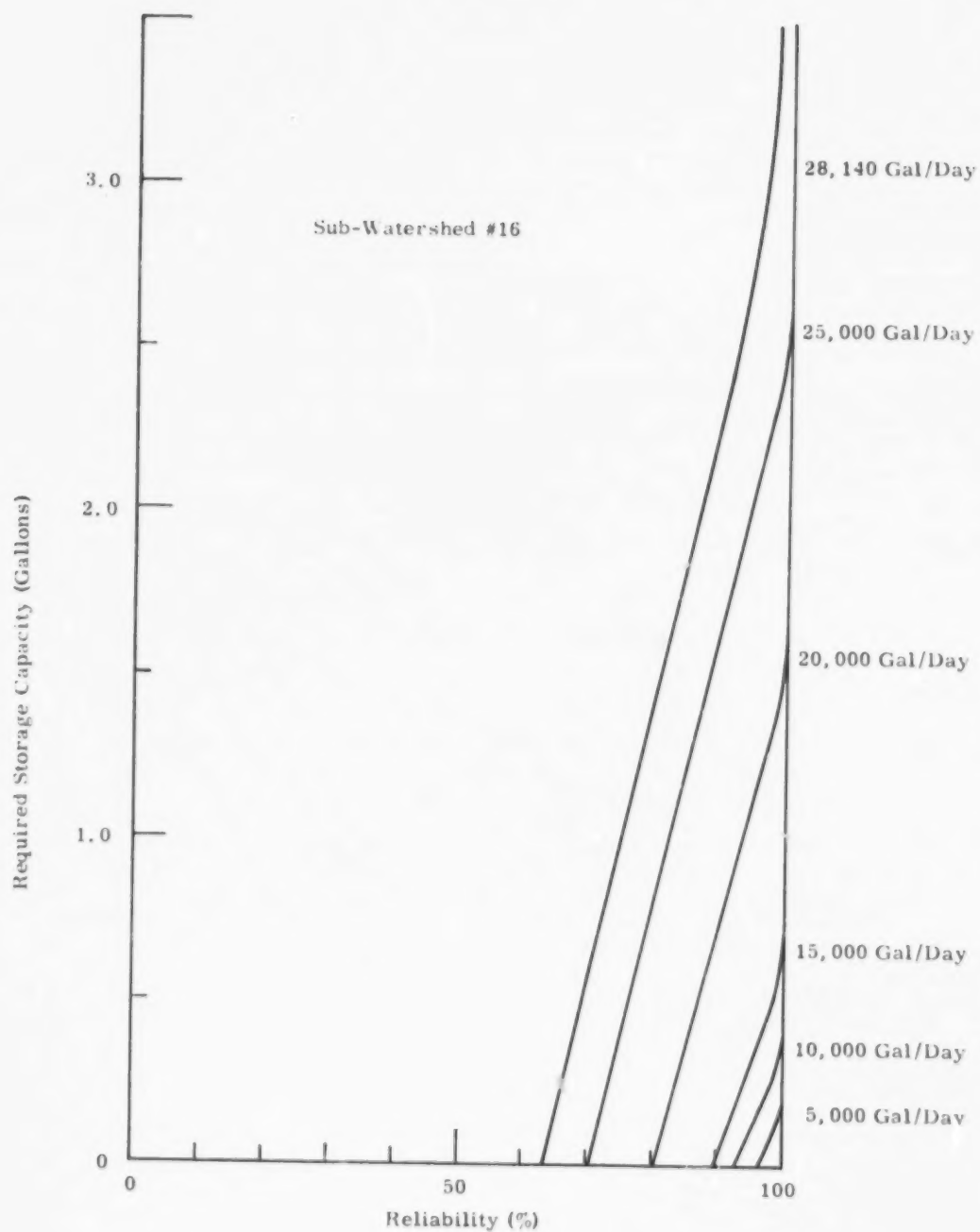


Figure 24. Storage/Reliability Relationship vs. Average Daily Demand

For purposes of the system study, therefore, it was assumed that trash would be allowed to enter the storage facility for later manual removal. The outlet from this facility, which would operate at flows much below storm flow rates, could be screened to exclude trash from the treatment works and downstream areas without creating appreciable maintenance problems.

An important aspect of water quality control along a natural watercourse is the protection of downstream uses through maintenance of minimum flows. In the case of Wilde Lake, it was considered desirable to protect existing stream bed ecology as well as maintain evaporation losses in Wilde Lake along with the release requirement imposed on the Wilde Lake dam by the Maryland Department of Water Resources. These factors will be taken into account in the development of operating criteria in a later section.

SEDIMENTATION

The operation of the storage facility as a sedimentation basin is of interest in sizing the facility. Large quantities of suspended solids must be removed from storm water in order to meet the minimum water quality standards described in Section V. In order to accomplish this objective, a reservoir may be operated as a combination storage and sedimentation facility, or a storage facility may be followed by a separate sedimentation device. Sedimentation will occur whenever the velocity of the storm flow is reduced by directing it into a structure or basin of large cross section, compared to the stream channel. Since the storage reservoir will function as a sedimentation basin regardless of its design criteria, the first approach was to analyze the effectiveness of this type of sedimentation device.

The unhindered settling velocity of free and discrete particles suspended in a liquid has been given (42) as:

$$v_s = \frac{2g}{C_D} \frac{\rho_s - \rho}{\rho} \frac{V}{A_c} \quad (22)$$

where C_D is the Newtonian drag coefficient, a function of particle geometry, fluid characteristics, and Reynolds number, and where:

- V = particle volume
- A_c = projected area of the particle
- g = acceleration of gravity
- ρ_s = mass density of the particle
- ρ = mass density of the fluid

If a velocity, v_o , is defined for a specific ideal sedimentation chamber in the following terms:

$$v_o = \frac{Q}{A} \quad (23)$$

where:

Q = overflow rate

A = surface area of chamber

then it can be shown that:

when $v_s > v_o$, the particle settles; and

when $v_s < v_o$, the particle does not settle.*

*(Ref 47, p. 63)

Since increasing the area, A , tends to reduce v_o , the fraction of particles having settling velocities such that they settle in a given chamber increases as A increases. Assuming that construction cost decreases with the volume of the sedimentation chamber, a reasonable design objective would be a chamber having larger surface area, but relatively shallow depth. Such a design might create problems with sludge storage and removal, as discussed below.

When the characteristics and size distribution of the suspended particles are known, design of a suitable sedimentation facility is a straightforward problem. In the case of storm water, this information is not known and probably varies widely with time. Published reports (30, 31, 33, 35, 37) and limited sampling have indicated the presence of colloidal and near-colloidal ($v_s \rightarrow 0$) solids in significant quantities. Hazen has calculated the settling velocity of various particles representative of those found in storm water as shown in Table 17.

TABLE 17. SETTLING VELOCITIES OF
SELECTED PARTICLES, AFTER HAZEN
(Ref. 48, p. 1474)

<u>Kind of Material</u>	<u>Particle Diameter (μ)</u>	<u>Settling Rate (cm/sec)</u>
Coarse sand	1000	10.0
Coarse sand	200	2.1
Fine sand	100	0.8
Fine sand	60	0.38

TABLE 17. SETTLING VELOCITIES OF
SELECTED PARTICLES, AFTER HAZEN
(Ref. 48, p. 1474) (Continued)

Kind of Material	Particle Diameter (μ)	Settling Rate (cm/sec)
Fine sand	40	0.21
Silt	10	0.015
Coarse clay	1	0.00015
Fine clay	0.1	0.0000015

Since the distribution of particle sizes is not known, a conservative approach to sedimentation facility design is indicated. Hydrologic investigations reported in Section IV estimated the mean runoff rate from sub-watershed number 14 at approximately 29.5 gallons per minute (gpm). A range of mean overflow rates might be considered in design of the sedimentation facility, but obviously none can be less than the mean runoff rate. Since reducing the overflow rate tends to reduce the area requirement for a given settling velocity (Equation 22), Table 18 represents the minimum areas where settlement can be achieved under ideal conditions for the particles indicated.

TABLE 18. MINIMUM SEDIMENTATION BASIN AREA REQUIREMENTS
FOR SELECTED PARTICLES, SUB-WATERSHED NUMBER 14

Kind of Material	Particle Diameter (μ)	Minimum Area Requirement (square feet)
Coarse sand	1000	0.20
Fine sand	100	2.50
Silt	10	133.56
Coarse clay	1	13,356
Fine clay	0.1	1,335,600

In practice, the area requirements would be considerably larger since the overflow rate would have to be greater and flow and velocity distributions would be greatly different than the ideal conditions.

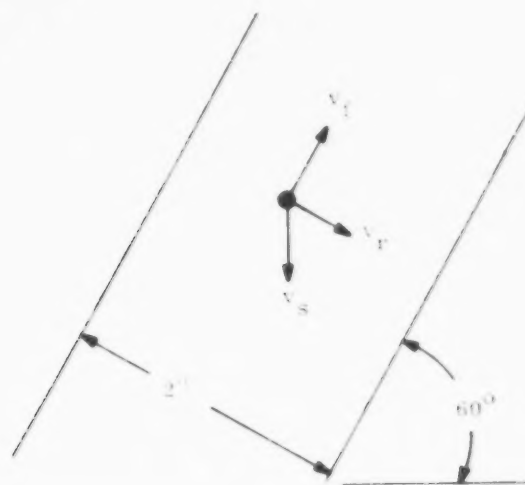
As a result of this evaluation, it was concluded that storage basins of the sizes that would be feasible in most locations would not alone produce

the desired water quality. Although the basins would serve to remove most of the larger size materials, the unfavorable velocity gradients that would exist during storm events would disturb the sedimentation process. This would cause the carryover of the smaller particles. For this reason, it was determined that supplemental treatment would be required to remove the small particles. However, the storage pond would still provide for the removal of a large portion of the solids and would reduce the sediment handling requirements of the supplemental treatment devices.

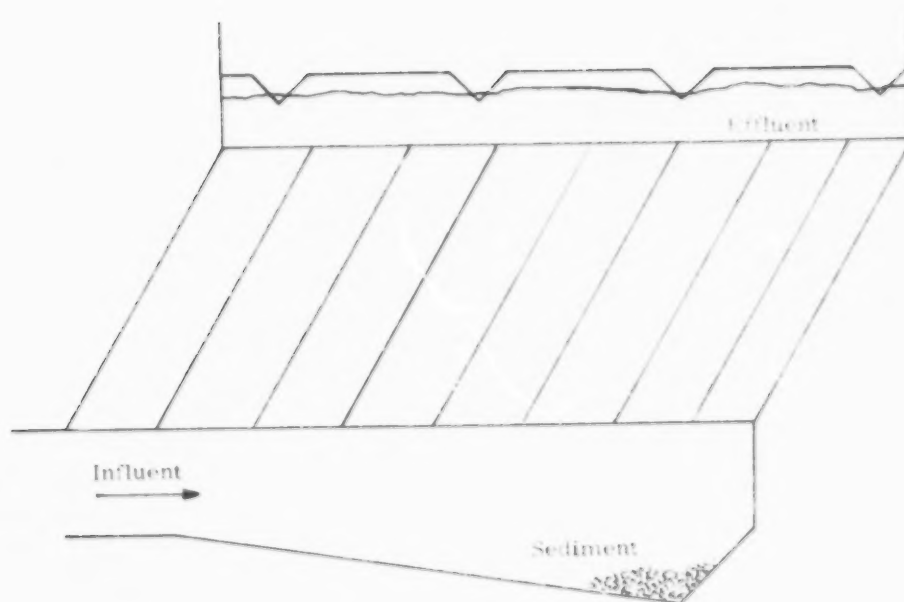
PRETREATMENT UNIT

Alternatives to the use of conventional sedimentation basins include the use of specialized sedimentation devices offering high effective-to-total surface area ratios, the application of chemical coagulants and coagulant aids to agglomerate small or light particles, and combinations of these techniques. Examination of settling requirements for particles below 1.0 micron in diameter led to selection of a combination approach - the use of an advanced sedimentation technique with chemical coagulants and/or coagulant aids. This facility, combined with chlorination of settled water, is referred to in later sections as the "Pretreatment" process. All runoff captured by the storage facility is subjected to this process, whether for subsequent final treatment or discharge to the stream.

Costs for the pretreatment phase were based on the use of a tube settler as marketed by Neptune-Microfloc, Inc., and described by Hansen and Culp (46). This device employs a series of small diameter plastic tubes inclined upward approximately 60 degrees from the horizontal. The flocculated water is permitted to pass upward through the tubes at controlled flow rates, typically between 3.0 and 5.0 gpm/square foot of cross sectional area. Figure 25 illustrates the principle of operation of the tube settler. Water moving up the tube at velocity v_f contains particles having a settling velocity v_s . The resultant velocity, v_r , tends to move the particles toward the tube wall, where they become trapped in a layer of particles previously removed. A two-inch diameter tube four feet in length can be operated at flow velocities as high as 2.84 gpm/square foot tube area and still permit a 10 micron silt particle to move completely across the tube. The corresponding loading on a conventional sedimentation basin is 0.015 cm/second or 0.221 gpm overflow rate per square foot of surface area (Equation 22) (see Table 17). This comparison is based on ideal conditions and does not take into account hindered settling conditions caused by the concentration of particles as they near the sludge collection zone. In the case of plain sedimentation, basins are commonly maintained at a minimum depth of five to eight feet to allow sufficient space above the sludge storage zone for hindered settling to occur in a low velocity boundary zone between the top layer, where flow is close to average overflow rate, and the static bottom layer. In the case of the tube settler, the steep inclination of the tubes causes the sludge to counterflow along the side of the tube as it accumulates, falling into a sump below the tube assembly. The tube settler configuration also requires influent and



Vector Relationships for Particle in Tube Settler



Typical Operation of the Tube Settler

Figure 25. Tube Settler - Principles of Operation

effluent plenums to distribute the flow to the tubes and to collect it after clarification. Figure 26 shows a comparison of the volume of typical sedimentation facilities designed to remove 10-micron silt particles at various overflow rates. The tube settler volumes include provisions for influent flume, influent plenum, sludge storage sump, and effluent plenum, while the conventional sedimentation volumes are based on an average depth of 5.0 feet to provide for sludge collection and storage. Inside partitions and supporting structures are included, but outside walls are not.

Other sedimentation devices, such as tray settlers, were investigated. Performance characteristics fell between conventional sedimentation and the tube settler, but sediment removal and maintenance problems were more severe than either of the two techniques compared above.

PHYSICAL AND AESTHETIC CONSIDERATIONS

Inherent in the concept of the Local Storage, Treatment, and Reuse of Storm Water system is the close proximity of the system elements to the properties which benefit from the system. In addition to engineering and economic considerations then, it is essential that the physical appearance of the facilities be consistent with the immediate environment. In the case of Columbia, the storage and pretreatment units must be located in the floodplains to permit gravity flow through the units. In general, the floodplains of the Wilde Lake watershed have been dedicated to public use as open space. They are typically lightly forested with some shrubbery and underbrush and are bounded on all sides by residential land uses. Any construction within these areas must be particularly unobtrusive in order to achieve acceptability within the community. The pretreatment units and constructed storage can be placed below grade and any necessary superstructure, i.e., hatchways, vents, etc., can be disguised with shrubbery, fences, etc. In the case of open pond storage, the design of the pond must insure that its appearance under all conditions of operation will be one of an attractive, natural pond. All construction must be confined to an area which will permit at least a fringe of trees along the perimeter, thus avoiding a conspicuous discontinuity in the open space. Subsequent cost estimates of constructed storage and pretreatment units are based on their installation entirely below grade, while open pond estimates provide for design features to insure their acceptability.

SIZING STORAGE FOR MULTIPLE PURPOSES

Earlier paragraphs described methods for determining required storage capacities for various conditions of water demand. It was found that for a given demand with a known proportion of lawn sprinkling uses, the storage requirement is a function of the reliability of the system or, stated differently, a function of the quantity of water drawn from the existing public water supply system in times of dry weather. Water quality considerations were found to affect operation policy more than

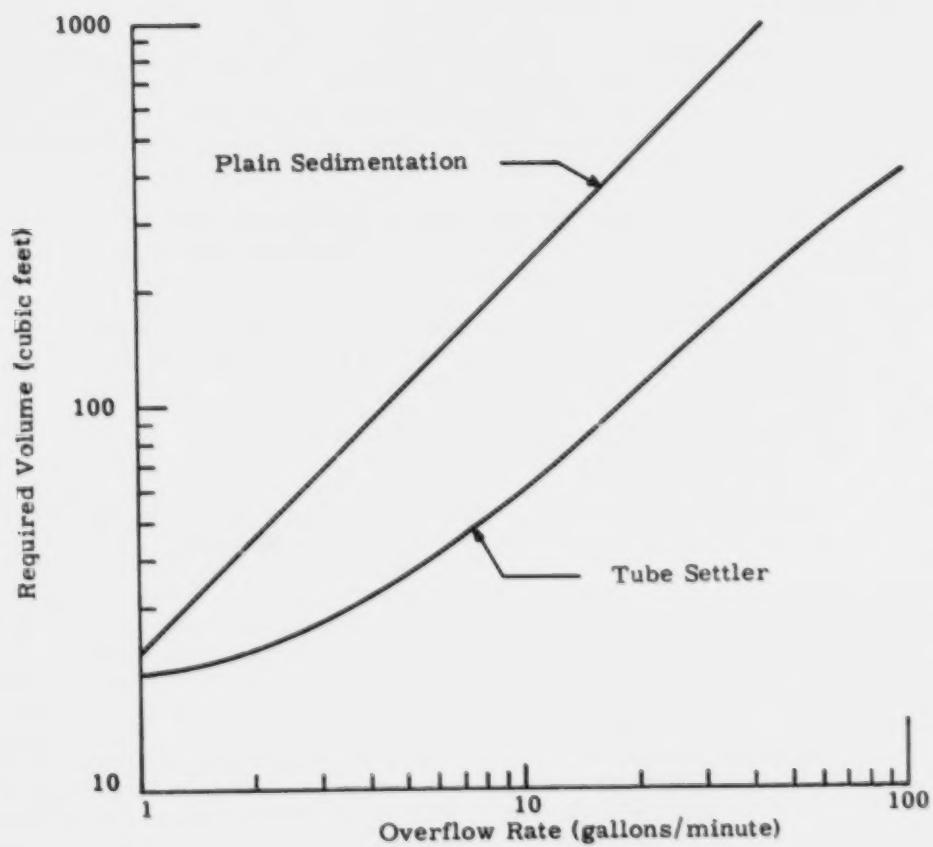


Figure 26. Required Volume vs. Overflow Rate for Removal of 10-Micron Silt Particle Under Ideal Conditions

design size, since meeting the standards by sedimentation in the reservoir was found to be impractical. The application of the pretreatment step following storage does, however, impose a different type of yield requirement on the storage facility, in that it must be provided with a constant flow for proper operation, although that flow need not be continuous. Physical and aesthetic considerations were examined and found to apply primarily to design criteria other than size, except that available land may limit the maximum attainable storage in a particular location.

Since the operation of the pretreatment unit appears to place a definable size requirement on the reservoir, that requirement will be evaluated first, then reviewed in the light of other requirements. Based upon previous discussions, the following assumptions are made with respect to the operation of the storage facility as a supply to the pretreatment unit:

1. The reservoir will be operated on a fill-and-draw basis, storage capacity calculated as the difference between minimum and maximum pools.
2. The highest proportion of total runoff feasible must be actually intercepted by the reservoir and subjected to pretreatment.
3. When outflow to the pretreatment unit occurs, it must be at a constant controlled rate.
4. Outflow need not occur at all times, e.g., intermittent operation of the pretreatment unit is permissible.

The proportion of the total runoff which is captured by the storage facility is a function of the volume of individual storms and the probability of available storage coincident with the occurrence of each storm. Since the reservoir is on a fill-and-draw cycle, the probability of available storage is a function of both the maximum storage capacity and the draw rate, or the rate of operation of the pretreatment unit. The analysis of reservoir performance will be in terms of recurrence intervals of storms and it will be initially assumed that storage is available to capture the entire volume of some design storm. When storms of larger volume or longer return interval occur, the excess quantity will be spilled, bypassing the pretreatment unit. Figure 27 shows rainfall-intensity-duration curves for storms of various recurrence intervals in Howard County, Maryland. From the definition of recurrence interval, the following identity exists:

$$E(n) = \frac{P}{T} \quad (24)$$

where:

$E(n)$ = expected number of storms meeting or exceeding conditions for a return interval T (years) occurring during period P (years)

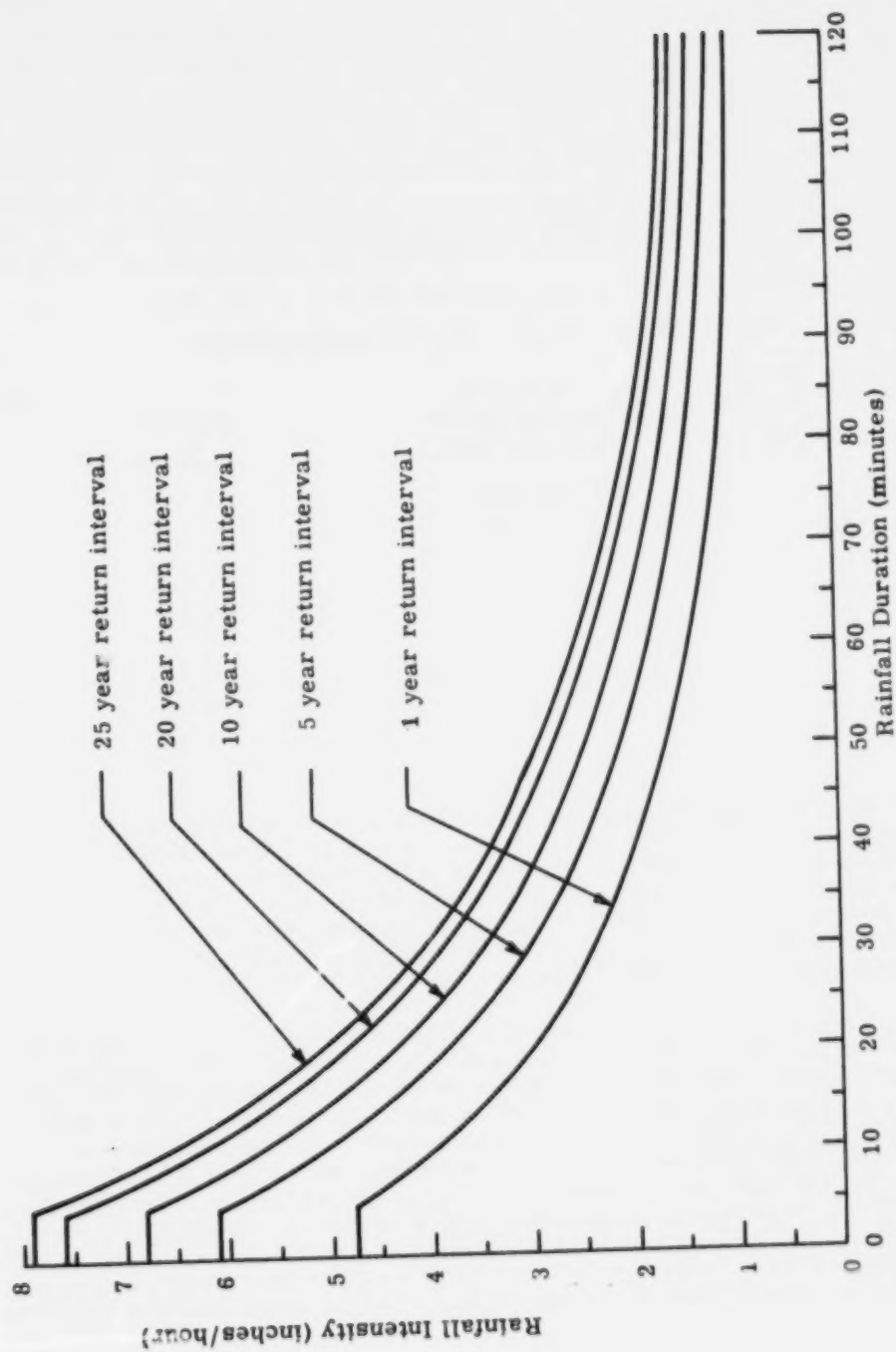


Figure 27. Rainfall Intensity-Frequency-Duration Curves,
Howard County, Maryland

Rainfall-intensity-duration curves can be used to generate runoff-intensity-duration curves for specific watersheds, using the methodology outlined in Section IV. By application of probability and expectation theory to these curves, the quantity of runoff occurring in excess of the storm volume corresponding to any selected return interval may be calculated. Table 19 lists the results of one such analysis.

TABLE 19. EXPECTED QUANTITY OF RUNOFF IN EXCESS OF SELECTED STORM VOLUMES

(Imperviousness = 0.30, m = (lag time) = 4 minutes)

(Total annual runoff = 220,307 gallons/acre)

Base Storm Return Interval (years)	Expected Excess Runoff (gallons/acre)	Excess (%)
1	16,160	7.34
5	5,050	2.29
10	3,066	1.39
20	1,857	0.84
25	1,584	0.74
50	962	0.44
100	579	0.26

Based on the most conservative calculation of storm volumes and on intensity-frequency-duration curves calculated to be conservative with respect to long-term records, a one-year storm volume storage capability can be expected to capture at least 92.66 percent ($100 - 7.34$) of the total runoff in a year. This value is dependent, however, on the availability of maximum storage at the beginning of each storm, a function of the release rate of the reservoir.

The release rate can be as low as the annual mean daily runoff rate, or $\bar{q}$. When this rate is assumed with a storage capacity equal to the volume of a one-year storm, analysis of the five-year synthetic hydrograph for sub-watershed number 6 reveals that an additional 3.01 percent of the runoff can be expected to spill as a result of the occurrence of storms whose volumes, although less than the one-year storm, exceed available storage. When the release rate is increased to $2.0\bar{q}$, the additional percentage of spill falls to 1.42 percent. Figures 28 and 29 show the approximate relationships between various releases or pretreatment rates and spill percentages for a range of storage capacities. It can be seen from these figures that a release rate of approximately $1.7\bar{q}$ in combination with a

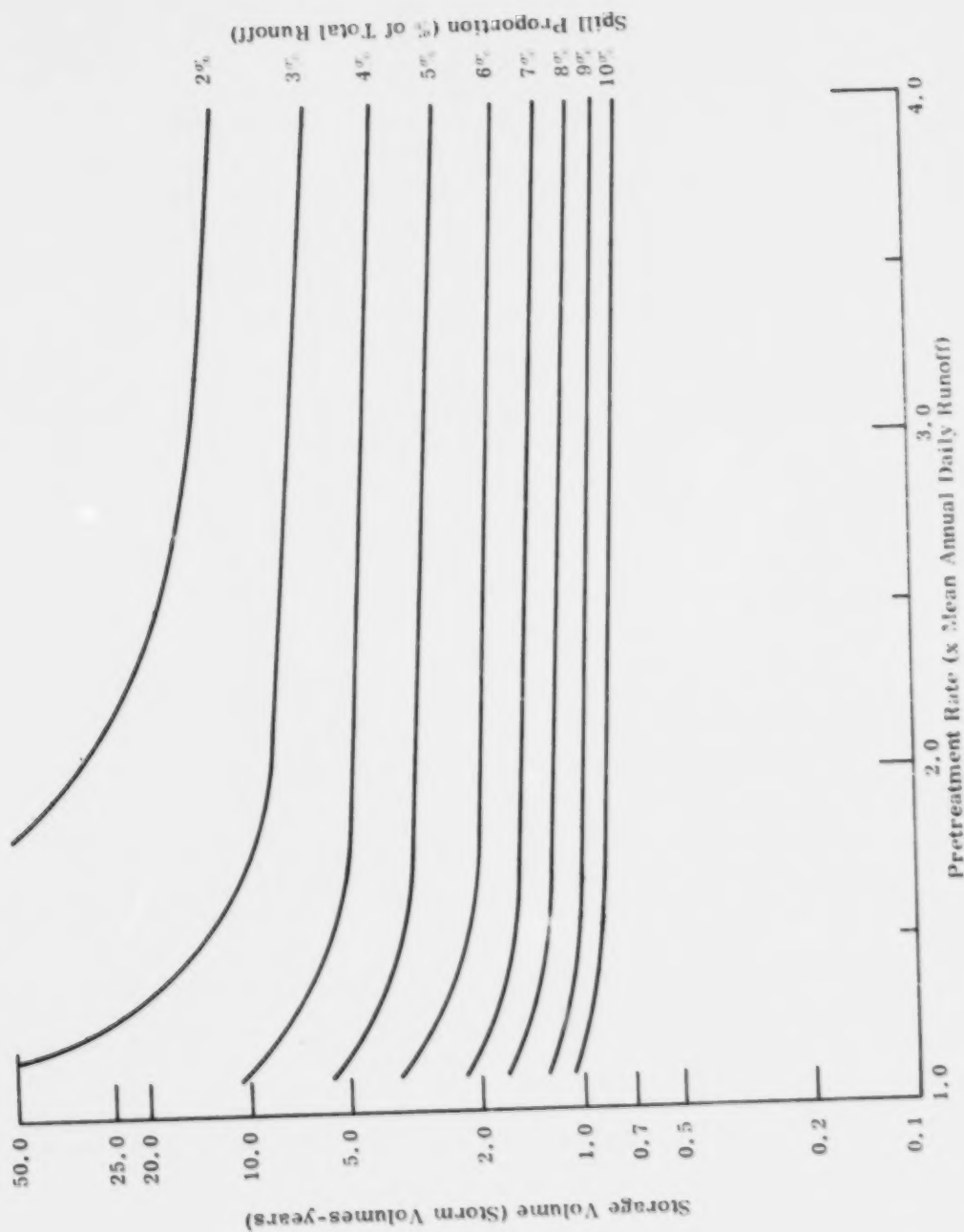


Figure 28. Storage/Pretreatment Rate Combinations vs. Spill Percentages

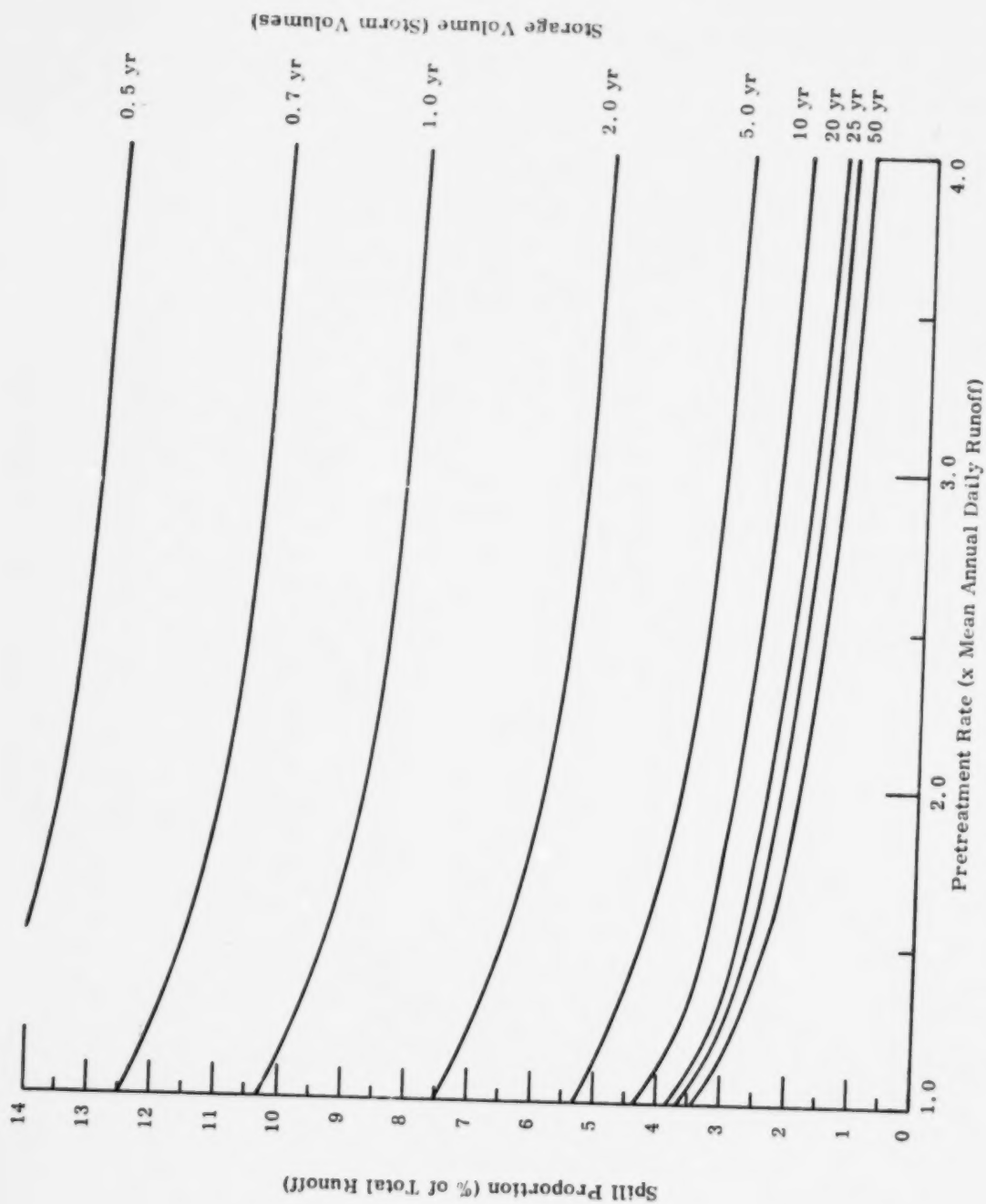


Figure 29. Pretreatment Rate/Spill Percentage Relationships vs. Storage

storage facility having a capacity equal to the volume of a one-year storm will result in approximately nine percent of the runoff being spilled without treatment. The remaining 91 percent will pass through the pretreatment unit at a controlled rate. Lowering pretreatment rate to approximately 1.1q increases the spill to 10 percent and greatly reduces storage requirements following the pretreatment unit where reuse is being practiced.

It is important to note that the water which does spill from the storage facility does so after passing through the storage structure where it has been detained for sufficient time to settle the larger and heavier solids. In view of the high quality of the balance of the water after pretreatment and the very infrequent nature of the spills, it was felt that restricting spills to 10 percent of total runoff was a realistic limit on the design and operation of the storage/pretreatment system. It would appear then, that a reservoir sized at the one-year storm volume operated in conjunction with a pretreatment unit controlled at a release rate somewhat greater than the mean runoff rate would process storm water within the water quality requirements of the system. A tradeoff obviously exists between spill volume and design criteria. Furthermore, a system having a given spill performance consists of a series of complex tradeoffs between storage capacity, release rate, and intermediate storage following pretreatment. Analysis of any of these tradeoffs, however, would require a quantitative estimate of the value of various levels of water quality, a task outside the scope of this study. Inherent in the study, however, is a method for inputting a value for water pollution control due to the study of the application of conventional methods of pollution control to the same watershed, reported in Section XVI. Both studies were directed to the same water quality requirement, and the 10 percent maximum spill requirement was made of both systems. The method employed to estimate spills was also common to both studies.

Having determined the storage requirement for effective water pollution control, this requirement must be weighed against other reservoir purposes. Figure 30 shows the storage/yield characteristics of sub-watershed number 16 plotted against the storage equivalent to the one-year storm volume. It can be seen that the maximum long-term demand of 25,327 gallons/day (mean runoff less 10 percent spill) can be sustained at the 80 percent reliability level. Similar relationships were found for each of the other sub-watersheds indicating that the water quality requirements on reservoir size result in a satisfactory reservoir from the water supply standpoint. The only other requirements found on reservoir size were those related to available land area and aesthetic considerations. Since these can only be determined by examination of a particular site with a particular size and construction type of proposed reservoir in mind, they were not considered until after the systems analysis effort was complete and conceptual designs were being produced from the results of the analysis, Sections X and XII.

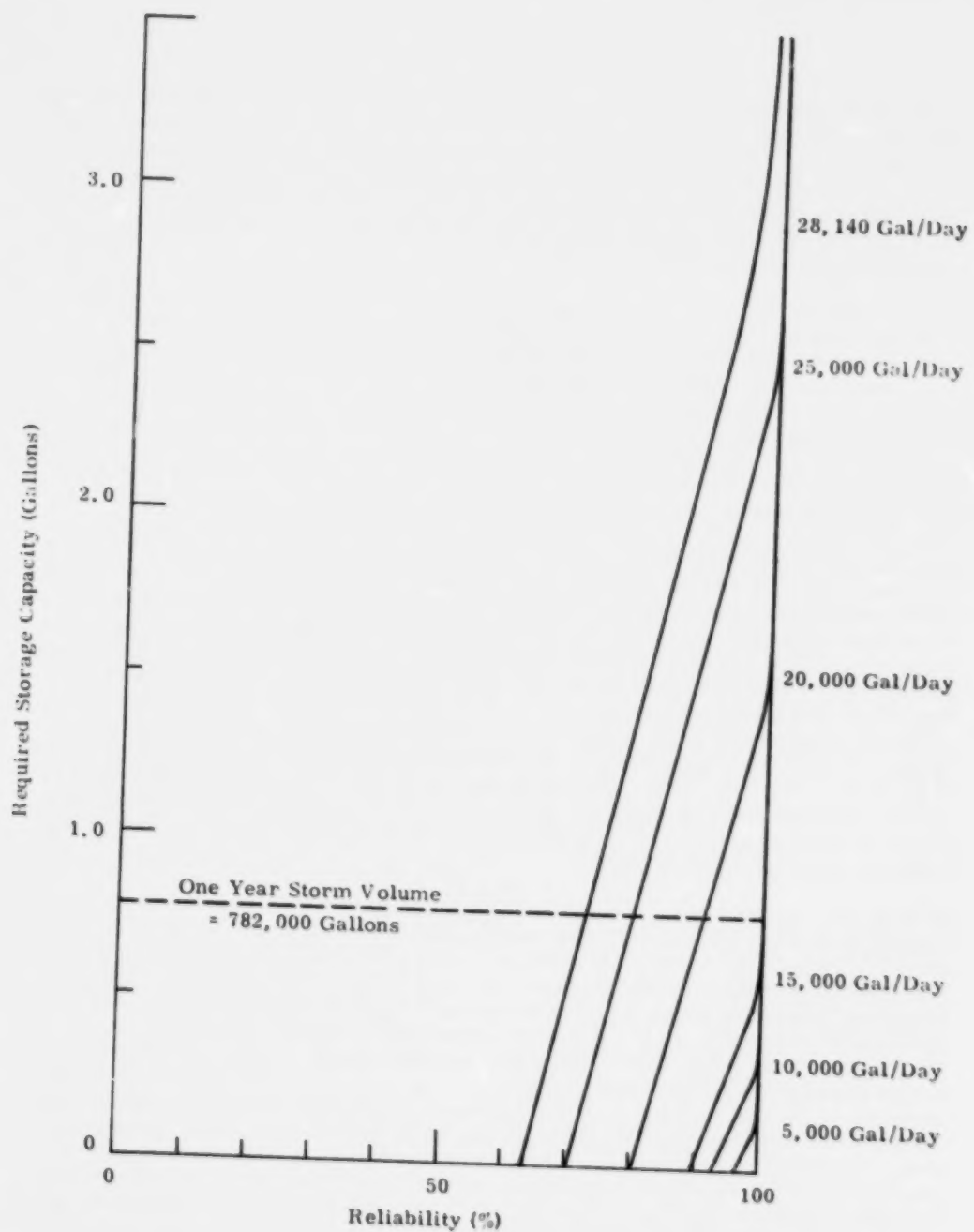


Figure 30. Storage/Reliability Characteristics vs. Yield, Sub-Watershed No. 16

As a result of a comparison of the various influences on reservoir and pretreatment unit design, the following criteria have been selected as adequate to protect all uses under consideration:

1. Net storage capacity of the reservoir shall be equal to the maximum volume for the sub-watershed or combination of sub-watersheds of a storm just meeting the one-year return interval intensity.
2. Release from the reservoir into the pretreatment unit will be controlled at a rate between 1.2 and 2.0 times the annual mean runoff rate.
3. The pretreatment unit will be capable of operating at 2.0 times the annual mean runoff rate.
4. No storage need be provided following pretreatment.

With respect to Criterion 2 above, it is noted that no discussion has been devoted to the effect of base flow on these criteria. Due to the use of the natural floodplains as collecting storm drains, it is likely that a significant amount of relatively constant flow will occur as a result of springs and seepage from the ground. This quantity is in excess of the volumes of runoff described in the analysis, although there is no ready way of predicting its quantity. Due to the steady nature of the flow, it places no particular requirement on storage, but will increase the required release rate and pretreatment rate by an amount equal to its flow rate. It is considered that in an area such as Columbia, the base flow would probably not be more than 30 percent of the mean runoff rate from storm water. Hence the requirement of 2.0 times the mean runoff rate for pretreatment design.

CONSTRUCTION, OPERATION, AND MAINTENANCE COSTS

In addition to the design criteria for storm water storage and pretreatment facilities discussed above, it was also necessary to develop information on the cost of constructing, operating, and maintaining these facilities. These data were developed parametrically in order to permit them to be used in the computerized system model. Estimates were made of the costs of storage and pretreatment facilities and of various capacities. These data were then plotted as smooth curves. The curves were then replaced by a series of linear approximations between the major inflection points. Tables were then prepared as inputs to the computer program so that cost data for any capacity system could be selected by linear interpolation between points from the tables. A further discussion of this technique and the development of the cost data used in the study are included in Appendix C. It is noted that these estimates are based on equipment and construction prices existing in the 1967-1968 time frame.

SEDIMENT REMOVAL

This study was based on a system of reservoirs operating in an established development which has restabilized. Accordingly, the sediment yields would not be excessive. For this reason, it was assumed that a majority of the solids would be removed from the pretreatment units and could be accomplished by pumping to tank trucks for transport and disposal. Periodically, sediment removal from the storage ponds would be required; however, the cost on a per gallon of runoff basis would be approximately the same as the pretreatment unit cleaning. The costs for storage basin and pretreatment unit cleaning would be considerably higher if the area was still under development or was undergoing continuing erosion due to the increased flows resulting from urbanization. The methods for maintaining storage ponds under these conditions are being considered as part of the demonstration program discussed in Section XIX.

SECTION VIII

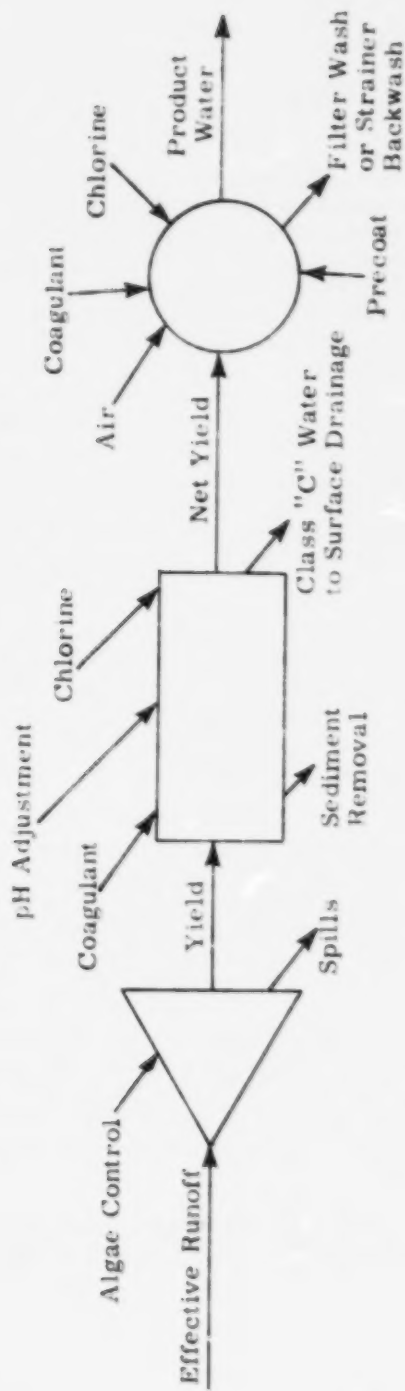
TREATMENT METHODS

Section VI described the wide range of pollutants and pollutant concentrations found in storm water by other investigators. The literature searches reported in Sections V and VI failed to locate reports of successful treatment of storm water immediately after collection from small watersheds. In the absence of a comprehensive storm water sampling and analysis program in the Columbia area, it must be assumed that storm water pollutants will be similar in number and concentration to those found in runoff from comparable areas. Table 10 in Section VI listed the pollutants whose presence is assumed and indicated the expected range of concentration of each. Specification of unit processes intended to achieve a given level of treatment must necessarily be conservative, due to the lack of reported field experience with similar problems. The use of systems normally used for treating water from conventional surface water impoundments was not considered directly applicable because of the random nature of certain pollutants and the small capacity of the individual treatment unit.

Section V described the four levels of treated water quality to be considered in the systems analysis. Various unit processes have been examined for each level, with attention to the maximum pollutant concentrations expected as well as the ability of the process to withstand severe shock loads. Table 20 lists the processes selected, indicating the treatment levels for which each is used. Figure 31 is a flow sheet showing the processes associated with each class of treatment.

Algae control is applicable only to those cases where storm water will be stored in an open facility. The term is intended to include all treatment steps necessary to prevent degradation of the stored water or the creation of a nuisance condition. Under some conditions, this might include steps to prevent mosquito or other insect breeding as well as the control of aquatic growths. Further discussion of these considerations can be found in Appendix C. After thorough analysis of the actual water to be treated, several of the processes listed might become unnecessary. Examples are pH adjustment and softening. Limited sampling conducted to date indicates that total hardness is low under base flow conditions and lower yet during storm flows, while pH varies between 6.3 and 8.3, depending on flow. Further sampling might indicate a need for some chemical adjustment of these parameters, however.

Descriptions of the systems that were assumed for treating the collected storm water as a function of quality and reuse classification are described in the following paragraphs.



STORM WATER STORAGE

- Algae Control* (all classes)

PRETREATMENT

- Flocculation (all classes)
- Sedimentation (all classes)
- pH Adjustment (all classes)
- Chlorination (C)

FINAL TREATMENT

- Straining (B)
- Softening* (AA, A)
- Flocculation* (AA, A, B)
- Filtration (AA, A)
- Carbon Adsorption (AA)
- Chlorination (AA, A, B)
- Aeration (AA, A)

*If required

Figure 31. Treatment Systems

TABLE 20. PROPOSED TREATMENT PROCESSES

Unit Process Type	Water Use Category		
	A-A	A	B
Raw Water Storage with Algae Control	X	X	X
Aeration	X	X	
Sedimentation	X	X	X
Flocculation	X	X	X*
Straining (with pre-coat)			X
Sand Filtration	X	X	
pH Adjustment	X	X	X
Softening	X	X	
Chlorination	X	X	X
Carbon Adsorption	X		

*IP required

TREATMENT TO CLASS "C" QUALITY

Class "C" treatment consists of mechanical trash screening, raw water storage with algae control, sedimentation, pH adjustment, and chlorination. Trash is excluded from the storm water collection system to a degree by the inlet design, which incorporates gratings and restricted openings. Smaller objects can enter the system, however, and parts of the subsequent collection system are open channels and natural watercourses, permitting further ingress of trash. Due to the difficulty of preventing a stoppage at the entrance to a storage facility, it is anticipated that no screening be provided at that point. This choice would depend on the difficulty of removing large objects from the storage basin itself. The point of withdrawal from the storage facility would then be screened to exclude trash from subsequent treatment processes. Since this screen would not be required to operate at storm flows, routine maintenance, as discussed in Appendix C, should permit satisfactory operation without the installation of mechanical cleaning devices.

Raw water storage might consist of any type of structure or basin, the only requirement being that its maximum elevation be at or below inlet elevation. This will permit storm flows to enter by gravity without adversely affecting the hydraulics of the storm water collection system. The possibility of pumping storm water at storm flow rates was excluded from the analysis for reasons of reliability. All available equipment designed for such intermittent, high rate service is electric motor driven. The correlation between electric power outages and severe thunderstorms is considered relatively high in an area of this type and such a joint occurrence would result in spills of large quantities of untreated storm water. Even though the facility could be designated to avoid flood damage from such an occurrence, the overall reliability was considered inconsistent with the water quality objectives of the study.

The decision to employ a specialized sedimentation device, or pretreatment unit, effectively eliminated sedimentation considerations from the design of the storm water storage facility. The criteria which were employed in sizing this facility are described in detail in Section VII. The storm water storage facility precedes the pretreatment unit and coagulants are introduced as the water is withdrawn from storage. The pretreatment unit is used as a point of application for pH adjustment chemicals, if required, and chlorine. Effluent from this combined operation is directed to the Class "C" water uses or allowed to overflow to the natural watercourse.

TREATMENT TO CLASS "B" QUALITY

Class "B" treatment consists of all the unit operations of Class "C," namely storm water storage with algae control (if necessary) and pretreatment, followed by a straining operation. This operation is the Microstraining process of the Glenville Kennedy Division of Crane Company, or equivalent. It consists of a fine mesh screen which is coated with an expendable

filter media, or precoat, so that effective removal of all particles 10 microns and larger can be achieved. Additional chlorine is added after this process in order to maintain an adequate residual throughout the distribution system. Following straining, the effluent is available for Class "B" or "C" water uses. Excess water will be permitted to overflow after the pretreatment process before entering the straining equipment. In this manner, distributed water will be at least Class "B," while excess water will be at least Class "C."

TREATMENT TO CLASS "A" QUALITY

Class "A" treatment consists of the unit operations of Class "C," storm water storage with algae control (if necessary) and pretreatment, followed by aeration, additional chemical coagulation, and mixed media filtration. Chemical additions and filter designs will be selected to ensure clarification of the effluent to 1.0 Jackson Turbidity Unit or better. This effluent, after rechlorination, will be distributed to Class "A," "B," or "C" uses. Excess water will be permitted to overflow after the pretreatment unit.

TREATMENT TO CLASS "AA" QUALITY

The treatment processes for Class "A" and Class "AA" are identical with the single addition of activated carbon adsorption in Class "AA" applications. This process removes tastes and odors as well as providing a buffer against occasional excessive concentrations of adsorbable organics and other dissolved materials. An activated carbon column utilizing a replaceable charge would be employed. The carbon charge would be periodically removed and disposed of or regenerated at a central facility. The capacity of the carbon column would be set at a value substantially in excess of anticipated requirements, in the interest of long life and security. Rechlorination would be practiced following the carbon treatment. As before, excess water will overflow after the pretreatment step.

TREATMENT SYSTEM COSTS

For each type of treatment described above, estimates were made of the equipment, facility, operating, and maintenance costs. These data were prepared parametrically using the same techniques used for the storm water storage facility and operating costs. The basis for the estimates of the treatment system costs and the input data used in the system analysis program are contained in Appendix D.

SUMMARY

The foregoing paragraphs do not represent suggested design for every circumstance. Actual configurations should be determined after suitable investigation of the characteristics and variations in the quality of

the storm water actually available. The processes described are only intended to be representative of adequate treatment under most local conditions and may well constitute over-design in many applications. Due to the supplemental nature of the water supply being evaluated, fail-safe controls can be employed to shut down the treatment facility in the event of malfunction without loss of supply to the connected demand. In this manner, considerable redundancy commonly a part of municipal water plant design can be eliminated and small package-type treatment units of low cost might be profitably employed. For purposes of the system analysis, however, no innovations have been considered. Every process discussed above has known performance characteristics consistent with the most difficult expected requirements.

SECTION IX

REUSE DISTRIBUTION SYSTEMS

The distribution system for storm water reuse is comprised of all the pipes, valves, meters, and appurtenances required to conduct the water from the point of treated water storage to the point of ultimate use. It includes connections to the existing public water system for makeup water with anti-backflow provisions at the interface between the two systems.

Pipe sizes throughout the distribution system have been selected for the estimated peak flows as well as reasonable ease of maintenance. Fire flows have not been assumed, since the public system will retain that function, consistent with the discussion in Section V. No pipes smaller than two inches have been assumed and the largest pipe diameters are six inches. Where a sub-potable distribution system serves a property, double meter vaults have been assumed with meters on both potable and sub-potable systems. Costs for sub-potable distribution systems within buildings include the additional costs of dual risers, extra fittings, etc., but do not include fixture costs. Sub-potable distribution systems have been designed for each of the distribution service areas described in Section V and laid out in such a way that any combination of continuous systems can be interconnected by simply operating valves (see Figure 32). This permits the evaluation of systems for various combinations of sub-watersheds without changing distribution costs, as well as providing for continued operation in the event of temporary failure of one system.

The transmission main from the treatment plant to the distribution piping has been calculated as a function of plant location only. The cost includes the public water system connection, which includes provisions for providing makeup water to sub-potable systems without the possibility of backflow into the public system. This might be accomplished by introducing public water into an open tank through a float-controlled valve located above the overflow, or, local regulations permitting, by one of several types of backflow preventer valves. In the case of potable reuse systems, the public connection is the point of injection into the public potable water distribution system. Some type of backflow preventing device might be employed here to isolate sections of the distribution system, depending on local conditions.

CLASS "C" AND "B&C" USES

When treated storm water is to be used to flush toilets only or to sprinkle lawns and flush toilets, a relatively simple dual distribution system is required. In addition to a sub-potable distribution piping in the streets, dual service connections and meters are required at each home, along with dual headers on the individual plumbing. Connections from the sub-potable system are made to the outside hose bibs for sprinkling use and to toilet fixtures throughout the building.

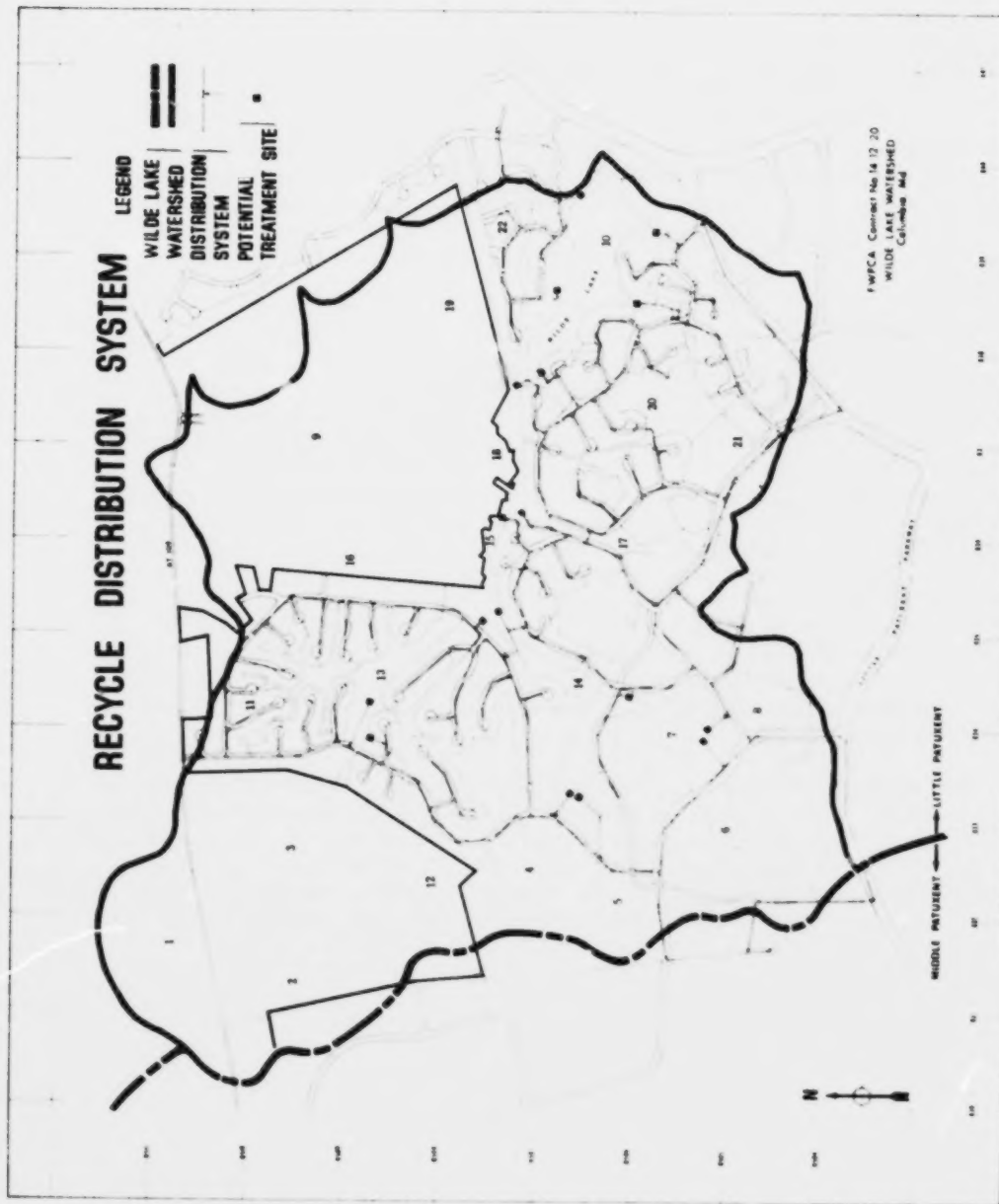


Figure 32. Distribution System Map - Wilde Lake Watershed

CLASS "B" USES

Reuse of treated storm water exclusively for lawn sprinkling requires a sub-potable distribution system and dual service connections, but minimizes the cost of plumbing alterations in each building. The sub-potable system is confined to the basement or ground level and is connected to outside hose bibs only.

CLASS "A", "A&B", "A&C", AND "A, B, &C" USES

These use combinations also require a sub-potable distribution system with dual service connections to each building. The sub-potable plumbing within the building, however, must be provided to each plumbing fixture except kitchen and bathroom sinks. In many homes, and particularly in apartment buildings, this will require virtual duplication of the entire cold water system. It is assumed that the hot water system could be split in multifamily units and perhaps in single family residences.

CLASS "AA, A, B, & C" USES

When storm water is treated to potable water quality, the distribution system is greatly simplified. The transmission line is provided from the treatment plant to the existing public water supply main where a metered connection is provided. The existing public system is then permitted to distribute the potable water to all water uses within the service area. It was anticipated that the service areas might be divided by check valves or division valves, but in practice this would probably not be desirable and the output of the treatment plant would supplement the supply throughout the system.

CONSTRUCTION COSTS

All costs for the distribution systems were estimated on the assumption that the necessary piping and plumbing would be installed following construction of the streets and buildings, and that no particular economies would be realized from concurrent construction with conventional facilities. Underground pipelines have been assumed to be cast iron and buried with a minimum of four feet of cover, with conventional valves and fittings. Individual building connections and plumbing alterations have been priced on a per unit basis. The in-place cost of cast iron pipe was taken at \$7.50 per foot of six-inch pipe and \$4.75 per foot of two-inch pipe. Table 21 lists the estimating figures used for building service connections and for interior plumbing. All figures include materials, labor, overhead, engineering, and construction contingencies and are 1967-68 prices. These prices are applicable to sub-potable distribution systems only and include provision for identification of pipes and valves by color coding, tagging, etc.

TABLE 21. CONSTRUCTION COSTS FOR SERVICE
CONNECTIONS AND INTERNAL PLUMBING FOR SUB-POTABLE REUSE

<u>Type of Occupancy</u>	<u>"C", "B&C"</u>	<u>"B"</u>	<u>"A", "A&B", "A&C", "A, B, & C"</u>
Single family residential, medium density	\$300/unit	\$200/unit	\$340/unit
Single family residential, low density	\$300/unit	\$200/unit	\$340/unit
Town houses	\$300/unit	\$200/unit	\$340/unit
Garden apartments	\$1000/20 units or fraction plus \$65/unit	\$1000/20 units or fraction	\$1000/20 units or fraction plus \$100/unit
Mid-rise apartments	\$80/unit	-----	\$100/unit
Schools, 500 pupils	\$1000/building	\$1000/building	\$1000/building
Commercial uses	individually estimated		

OPERATION AND MAINTENANCE COSTS

Operation and maintenance of sub-potable distribution systems is primarily composed of occasional repair of distribution mains, the portion of the service connections in the public right-of-way, and routine inspection and reading of the individual meters. Only the incremental cost of reading the additional meter was computed, since it would be read simultaneously with the existing potable water meter. These costs were estimated on a yearly basis as 0.006 times the construction cost of the system. The formula employed to convert these costs to daily costs is:

$$\text{DACOS} = \text{ATOT} (0.006/365) \quad (25)$$

where:

ATOT = annual total construction cost of distribution system

DACOS = daily total operation and maintenance cost of system

The portion of the service connection which is located on private property and the internal plumbing would be maintained by the individual property owner in the same manner as the existing conventional plumbing which it supplements.

INPUT TO SYSTEM MODEL

The construction cost for each service area, or sub-watershed, was computed for each group of reuse classes as described in the preceding paragraphs. These values were used as input to the computer program, permitting the selection of the proper costs for any sub-watershed or, by adding costs for component areas, the proper value for any combination of sub-watersheds. The costs of transmission lines from treatment plants were computed separately for each treatment plant location. These values are shown as Tables 22 and 23, respectively.

TABLE 22. CONSTRUCTION COST VS. SERVICE
AREA - DISTRIBUTION SYSTEM

Sub- Watershed	Construction Cost for Reuse Level (Dollars)					
	"A"	"B"	"C"	"A", "B"	"A", "C"	"B", "C"
1	17,540	14,600	16,700	17,540	17,540	16,700
2	20,990	18,050	20,150	20,990	20,990	20,150
3	10,330	9,210	10,010	10,330	10,330	10,010
4	39,060	27,300	35,700	39,060	39,060	35,700
5	45,800	30,120	41,320	45,800	45,800	41,320
6	269,920	163,400	238,930	269,920	269,920	238,930
7	141,760	85,900	125,520	141,760	141,760	125,520
8	54,500	31,900	47,720	54,500	54,500	47,720
9	151,300	136,600	147,100	151,300	151,300	147,100
10	135,650	92,250	122,575	135,650	135,650	122,575
11	80,890	63,950	76,050	80,890	80,890	76,050
12	44,180	36,200	41,900	44,180	44,180	41,900
13	54,450	41,850	50,850	54,450	54,450	50,850
14	110,820	82,000	102,300	110,820	110,820	102,300
15	22,460	18,120	21,220	22,460	22,460	21,220
16	82,760	70,300	79,200	82,760	82,760	79,200
17	108,720	78,660	104,680	108,720	108,720	104,680
18	31,320	24,840	29,440	31,320	31,320	29,440
19	56,360	50,200	54,600	56,360	56,360	54,600
20	101,240	78,840	93,440	101,240	101,240	93,440
21	83,330	50,910	72,415	83,330	83,330	72,415
22	33,860	26,020	31,620	33,860	33,860	3,162

TABLE 23. CONSTRUCTION COSTS VS. SERVICE
AREA-TRANSMISSION LINES

<u>Treatment Plant Location (Sub-watershed)</u>	<u>Construction Cost (Dollars)</u>
1	3000
2	3000
3	4500
4	2250
5	5550
6	2630
7	750
8	1800
9	2250
10	1950
11	3900
12	1500
13	750
14	1500
15	2400
16	3750
17	4280
18	2850
19	1730
20	1580
21	4280
22	1500

SECTION X

SYSTEM MODEL

The system study of the local storage, treatment, and reuse of storm water concept has been outlined in Section III. The system was defined in Figure 1. The objective function was stated as:

1. Water released to surface drainage from the Local Storage, Treatment, and Reuse of Storm Water system shall meet or exceed stated effluent standards.
2. This condition will be achieved at the lowest net system cost, or maximum net system benefit.

Maximum net system benefit was, in turn, defined as:

$$\max (\Sigma B_a - \Sigma C_a - a \Sigma C_f)$$

where:

- B_a = marginal annual benefits
- C_a = marginal annual operating and maintenance costs
- C_f = marginal fixed construction and project costs
- a = capital recovery factor (Equation 3)

The effluent standards required by the first part of the objective function were developed in Section VI and shown as Table 10. The constraints on the objective function were identified and investigated and the results reported in preceding sections. All information obtained concerning these constraints was prepared in a suitable form for input to a digital computer code for maximization of the objective function subject to the constraints.

A review of the prior sections indicates a large number of possible physical systems, all of which would satisfy the stated physical, technological, and institutional constraints. Twenty-two sub-watersheds were considered, and a local system could be placed in each of them. In this manner, the total watershed could be served by twenty-two local reuse systems, making up the overall system for the watershed. This is only one of many ways to meet the requirements of the watershed, however. There are many points in the watershed where two, three, or more sub-watersheds can be collected at one point. Figure 33 shows the flow pattern of the various sub-watersheds and can be used to determine feasible combinations of multiple sub-watersheds. In these cases, the combined sub-watersheds are treated as a single area and have a single local reuse facility. As many as eighteen sub-watersheds can be combined into a single collection area, and four which discharge directly

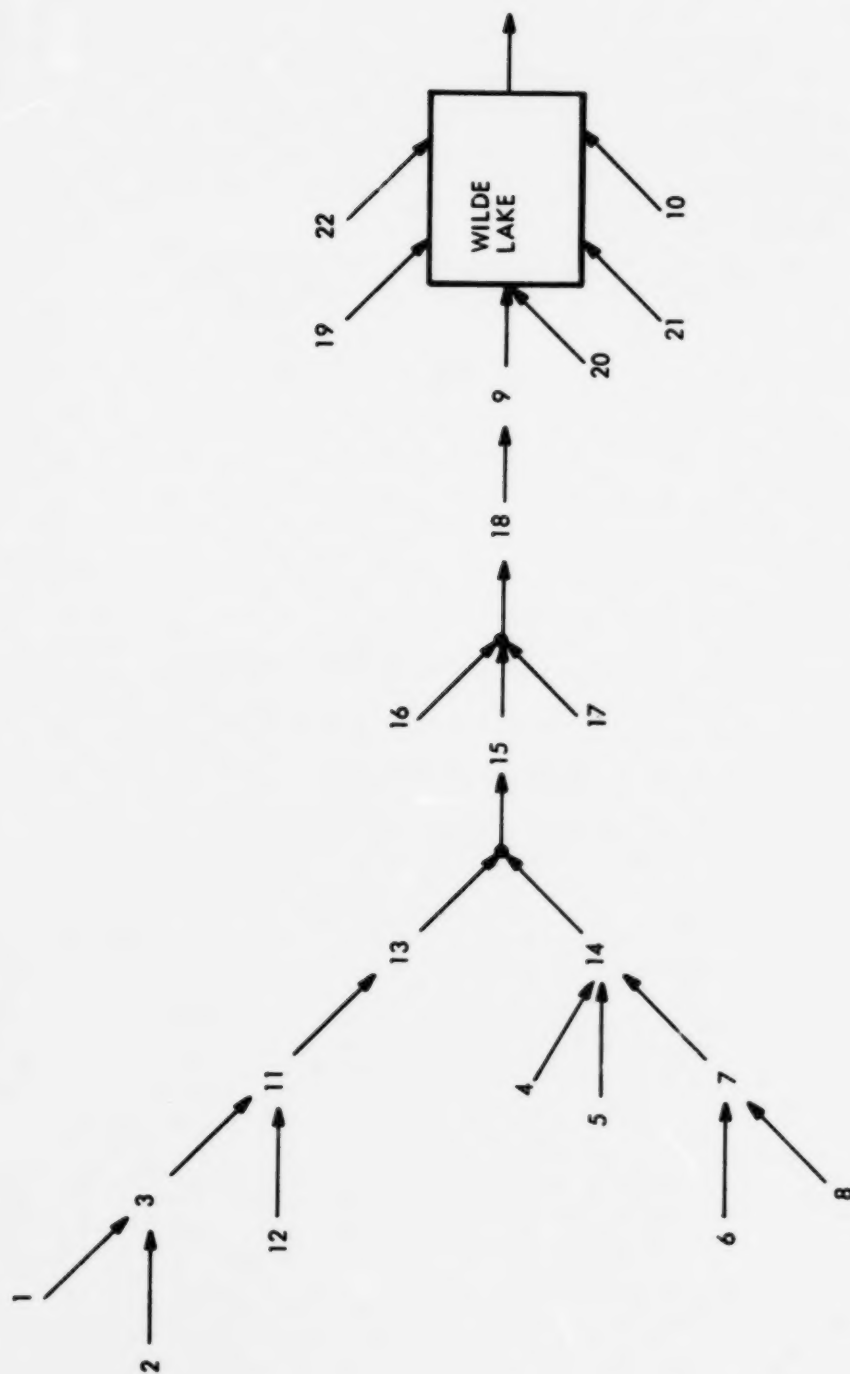


Figure 33. Wilde Lake Drainage Area Sub-Watershed Flow Pattern

into Wilde Lake must remain independent. An exhaustive analysis of the flow diagram would reveal that more than 10,000 combinations of various sized collection areas are possible. Fortunately, many of these are trivial, and others present varying degrees of difficulty in terms of physically intercepting the storm flows at the required points. Close examination of the watershed resulted in 173 workable combinations of service areas, each comprised of some number of sub-watersheds, ranging from one to six. In view of the study objective of evaluating local storage, treatment, and reuse of storm water, an upper limit of six sub-watersheds, or about 350 acres, was set for the initial attempt at system optimization. It was planned to reconsider this decision after the first trial, but the results, reported below, indicated that the optimum had been located within the range selected.

Each of the 173 overall systems is made up of a number of sub-systems, namely, the various storage units, pretreatment units, final treatment units, treated water storage units, distribution systems, etc., that comprise the facilities in each collection area. The storm water might be treated to any of four qualities and the distribution system might be constructed to connect to any of eight different combinations of water uses. Within a given overall system, the individual facilities might vary from each other. Each of these possible sub-systems and overall systems must have construction, operating, and maintenance costs calculated, adjusted to annual costs, and subtracted from the net benefit that would be derived from the overall system. When the remainder from this calculation is available from each of the possible systems, the optimum system can be identified and described. Due to the great complexity and magnitude of the calculations required, the system optimization was performed by a digital computer. The following sections describe the preparation of inputs to the computer program, the operation of the program, and the type of outputs obtained. The system optimization results will be described, and final investigation of alternatives reported.

DESIGN INPUTS

Storage Facilities

In order to insure that the systems under analysis all correspond to the range of technologic and physical possibilities, the essential design criteria have been determined and must be included as inputs to the system model, prior to optimization wherever possible. In the case of storm water storage, various types of storage were considered which might have varying applicability in different locations. Steel and concrete tanks, both buried and at grade, were considered, along with open ponds. The system model was arranged for a sub-optimization of storage facilities, based on economic factors, which is described later. In the event that local conditions prohibit a free choice among the alternatives, the program can be instructed to choose either open ponds, buried tanks, or nonburied tanks as the only possible construction method. It can also

be instructed to choose only steel tanks or only concrete tanks, when tanks are being considered.

Section VII described the analysis performed to develop sizing criteria for reservoirs. It was determined that the various reservoir purposes could be served adequately by sizing the storm water storage facility at the maximum expected volume of a storm just meeting the one-year return interval criteria. The one-year storm volume has been calculated for each sub-watershed, using the results of the hydrological analysis reported in Section IV, and these values are listed as Table 24.

TABLE 24. EXPECTED MAXIMUM VOLUME
OF ONE-YEAR STORMS

<u>Sub-watershed</u>	<u>One-year Storm Volume (gal)</u>	<u>Sub-watershed</u>	<u>One-year Storm Volume (gal)</u>
1	460, 800	12	594, 200
2	586, 500	13	696, 800
3	530, 000	14	1, 030, 000
4	341, 600	15	236, 400
5	530, 500	16	782, 000
6	1, 104, 000	17	1, 130, 000
7	803, 200	18	290, 000
8	625, 900	19	923, 500
9	1, 460, 000	20	1, 203, 000
10	1, 258, 000	21	690, 000
11	854, 000	22	358, 400

Since it was originally anticipated that storage volumes would be a function of demand, the systems analysis computer program was arranged to test the service area demand against the available supply before computing storage volume. The total runoff expected from each sub-watershed was calculated by methods described in Section IV. Further analyses described later disclosed that, given the design criteria selected for the Wilde Lake watershed, 90 percent of the total runoff will be available for reuse at

release rates of at least $1.13 \bar{q}$. Accordingly, the total demand of any service area under consideration can be compared to 90 percent of the total runoff of that same area to determine the availability of the required volume of treated water. When the demand exceeds the available supply, it is reduced to equal that supply. This insures that later calculations of system costs, benefits, etc., refer only to the portion of the demand that will be satisfied by treated storm water. The balance of the demand will be satisfied by the public water supply system and will not figure in the calculation of costs or benefits. When the demand does not exceed the available supply, subsequent calculations will be based on the actual demand. Table 25 lists the available supply for each of the sub-watersheds, equal to 90 percent of total runoff.

TABLE 25. EXPECTED AVAILABLE SUPPLY FROM STORAGE

<u>Sub-watershed</u>	<u>Available Supply (gal/day)</u>	<u>Sub-watershed</u>	<u>Available Supply (gal/day)</u>
1	8,946	12	18,090
2	11,565	13	24,989
3	10,064	14	38,340
4	11,807	15	9,129
5	19,080	16	25,327
6	43,513	17	50,305
7	33,915	18	9,630
8	26,468	19	38,520
9	43,994	20	50,193
10	61,997	21	28,734
11	30,552	22	13,410

When the service area under consideration consists of more than one sub-watershed, storage volume and maximum demand can be found by adding the values associated with each of the component sub-watersheds on the tables above.

Pretreatment

Operating criteria developed in Section VIII for the pretreatment unit stipulated that it must be capable of operating at twice the mean rate of runoff. This can be accomplished within the system model by inputting a table of total annual runoffs from each sub-watershed (Table 26) and using these figures to determine costs for the pretreatment unit. Furthermore, when the total system of sub-watersheds is analyzed, each downstream pretreatment unit must process twice the mean runoff from its own watershed as well as the total of the treatment rates of all the upstream units. The cost figures used for determining construction and operating cost of pretreatment units (Tables D-1 and D-7) are based on total runoff and treated water demand, respectively, and take into account the operating rule suggested for the unit. Construction costs are calculated to provide the necessary capacity, using the total annual runoff from Table 26.

TABLE 26. TOTAL ANNUAL RUNOFF VS. SUB-WATERSHED

<u>Sub-watershed</u>	<u>Total Annual Runoff (gallons)</u>	<u>Sub-watershed</u>	<u>Total Annual Runoff (gallons)</u>
1	3, 626, 640	12	7, 332, 120
2	4, 693, 608	13	10, 133, 568
3	4, 073, 400	14	15, 528, 105
4	4, 790, 547	15	3, 700, 224
5	7, 732, 769	16	10, 259, 712
6	17, 653, 330	17	20, 388, 024
7	14, 006, 578	18	3, 905, 208
8	10, 740, 789	19	15, 636, 600
9	17, 817, 840	20	20, 351, 232
10	25, 123, 680	21	11, 652, 552
11	12, 377, 880	22	5, 439, 960

Final Treatment

No physical or technological constraints are placed on final treatment facilities, other than their categorization into four classes of treatment, as discussed in Chapter VIII. Class "C" consists of pretreatment only and only a portion of its operating costs are ascribed to final treatment. Classes "B," "A," and "AA" are composed of pretreatment and various levels of additional treatment. In all cases, demand is selected as the parameter for determining costs, and the demand is tested prior to calculation of treatment costs by the method described above in the paragraph headed "Storage Facilities."

Pumping Facilities

Pumping facilities are sized by adjusted treated water demand and are constrained only by cost.

Treated Water Storage

As in the case of stormwater storage, a number of storage configurations are analyzed. Steel and concrete tanks, above or below grade, as well as elevated tanks, standpipes, and hydropneumatic tanks are all estimated for each application, unless instructions are input to do otherwise. The program can be instructed to select any of the storage configurations without reference to the others. Treated water storage tanks are sized as a function of demand by the relationships shown as Equations D-1, D-2, and D-3.

Distribution System

The various physical constraints on distribution systems were discussed in Section IX and were taken into account in developing distribution system costs. Distribution costs are a function of the sub-watershed under study and the level of reuse being considered. Sub-watersheds can be combined by simply adding distribution costs. The transmission line to the treatment plant itself is listed separately (Tables D-10 and D-11) and is a function of treatment plant location only. Where one treatment plant serves two combined sub-watersheds, the two distribution costs are summed, but only the transmission line costs for the actual location of the plant apply.

SYSTEM COSTS

Storage Facilities

Construction, operating, and maintenance costs have been developed for five different types of storm water storage facilities and presented as

tables of cost/capacity relationships. Capital costs are composed of land cost, site preparation costs, burial costs where applicable, and erection costs. Appendix C lists these costs. Average cost of land in the Columbia area was taken as \$12,000 per acre. Later discussions with the developer of Columbia led to a decision to confine facilities to land already dedicated to public use, land not usable for other purposes due to its function as natural floodplain. Since the cost of this land was not chargeable to the local reuse system, land costs were set at \$0.00 per acre for purposes of systems analysis. An auxiliary computation indicated that total system costs were reduced approximately seven percent by this assumption. Land area, for purposes of computing land costs and site preparation costs, is found from a land area table (Table C-2) in the case of open ponds, and computed from capacity and H/D ratio (Table C-3) in the case of tanks. Operation and maintenance costs are made up of sediment removal costs and metal tank maintenance. Methods for computing these costs are given in Appendix C. After evaluation of all component costs, the total system cost of a storm water storage facility of any desired configuration can be expressed as:

$$C_j = a(L_j + P_j + B_j + E_j)/365 + (R_j + M_j) \quad (26)$$

where:

- C_j = total system cost of storage facility of configuration j (dollars/day)
- a = capital recovery factor (Equation 3)
- L_j = land cost for storage facility of configuration j (dollars)
- P_j = site preparation cost for storage facility of configuration j (dollars)
- B_j = burial cost, where applicable (dollars)
- E_j = erection cost (dollars)
- R_j = silt removal cost (dollars/day)
- M_j = tank maintenance cost, where applicable (dollars/day)

All system costs were calculated as dollars/day to simplify the output format. The annual rate of interest was taken at the 1967 level recommended by the Water Resources Council for evaluation of water resources projects - 0.032 or 3.2 percent annually. It was recognized that various persons and agencies interested in the storm water reuse concept will have a range of interest figures which they consider appropriate to the analysis. A common approach is to select a rate based on the current

cost of new capital to the agency considering construction of such a facility. This might range from 4.5 percent for the Federal Treasury (1967) to approximately 5.0 percent for most local governments in late 1967 or early 1968, to perhaps as much as 7.0 percent for a private developer. Since the local reuse system is being compared to a conventional approach which is characterized by relatively high capital costs and low operating costs as compared to the study concept, the selection of a high interest rate would tend to penalize the conventional central system. Accordingly, the 3.2 percent figure was selected as the lowest interest rate that might be considered for the analysis, a conservative assumption from the standpoint of evaluating the local reuse concept. The amortization period is likewise a subject of some difference of opinion. When such considerations are influenced by financing details, the life of the bonds issued to fund the project is a common criterion - frequently about 30 years. From an economic sense, the estimated useful life of the facilities might be used, preferably each individual unit but possibly the average useful life of all facilities. Fortunately, in this case no substantial disagreement existed between the two approaches. The estimated useful life of all facilities under consideration is considered to average approximately 30 years. For this reason, 30 years was selected as the period of amortization.

The methods employed to calculate the total system cost on a dollar/day basis of each of five possible configurations of storage have now been outlined. Following these calculations, a sub-optimization is performed for storm water storage configuration. Unless constrained to fewer choices by the options described above, the five system costs are compared and the lowest total system cost is selected. Results of the other four computations are destroyed and the selected system costs are retained as the final costs of storm water storage.

Pretreatment

Appendix C described the development of capital costs for the pretreatment unit. Total annual runoff is obtained for a given watershed from Table 26, and the pretreatment cost for this capacity is found by linear interpolation of Table D-1. Sediment removal costs are computed as part of the storage facility costs, and other operating and maintenance costs are combined with final treatment costs. Total system cost of the pretreatment unit is defined as:

$$C_p = a(S_p)/365 \quad (27)$$

where:

$$\begin{aligned} C_p &= \text{total system cost of pretreatment unit (dollars/day)} \\ S_p &= \text{construction cost of pretreatment unit (dollars)} \end{aligned}$$

Final Treatment

Final treatment costs are composed of construction and operating costs, each computed vs. daily treated water demand for four different levels of treatment. Costs for given demand and level of treatment can be obtained by linear interpolation of Tables D-3 and D-7 with respect to demand, using the sections pertaining to the treatment level selected. In this manner a construction cost and an operating cost are selected, and they are combined in the following manner:

$$C_t = a(S_t)/365 + O_t \quad (28)$$

where:

- C_t = total system cost of final treatment (dollars/day)
- S_t = construction cost of treatment unit (dollars)
- O_t = operating and maintenance cost of treatment unit (dollars/day)

Pumping Facilities

Construction costs of pumping facilities are obtained by linear interpolation of Table D-5. Operating and maintenance costs have been combined with final treatment costs. Total system costs for pumping are found by the following expression:

$$C_h = a(S_h)/365 \quad (29)$$

where:

- C_h = total system cost of pumping (dollars/day)
- S_h = construction cost of pumping facility (dollars)

Treated Water Storage

Seven different configurations of treated water storage facilities were analyzed and costs developed for each of them. The types of facilities examined include standpipes, elevated tanks, hydropneumatic tanks, ground level steel and concrete tanks both above and below grade. Construction costs for each of these are made up of land cost, site preparation costs, burial costs (where applicable), and erection costs. Land areas for land costs and site preparation costs are computed from the tank capacity and the H/D factor. Costs are summarized in Tables C-6, C-7, and C-9. H/D factors appear in Tables C-3, C-4, and C-5. Tank

maintenance costs consist of periodic painting and maintenance of metal tanks and are found by the methods described in Appendix C. As in the case of storm water storage, a sub-optimization was performed for each case by computing the total system cost for each tank (unless constrained by the instructions described above) as follows:

$$C_k = a(L_k + P_k + B_k + E_k)/365 + O_k \quad (30)$$

where:

- C_k = total system cost of treated water storage facility of configuration k (dollars/day)
- L_k = land cost for storage facility of configuration k (dollars)
- P_k = site preparation cost for storage facility of configuration k (dollars)
- B_k = burial cost, where applicable (dollars)
- E_k = erection cost (dollars)
- O_k = operation and maintenance cost (dollars/day)

The total system costs for the five possible configurations are compared and the least cost alternative is selected and its costs retained. Costs for the other approaches are not retained.

Distribution System

Costs for distribution systems for each sub-watershed and each level of reuse are input directly to the computer program. These costs are tabulated in Table 21. Service areas made up of several watersheds are calculated by adding the appropriate entries from the component sub-watershed costs. The transmission line to the treatment plant is listed separately on Table 22 and is added to distribution cost. Operation and maintenance cost is a function of total distribution capital cost and is calculated by Equation 25. Total system cost for distribution is calculated as follows:

$$C_d = a(S_d + S_c)/365 + O_d \quad (31)$$

where:

- C_d = total system cost of distribution system (dollars/day)
- S_d = construction cost of distribution system (dollars)

- S_c = construction cost of transmission line (dollars)
- O_d = operation and maintenance cost of distribution system (dollars/day)

FEASIBLE SYSTEMS

Figure 33 illustrates the flow pattern of the 22 sub-watersheds in the Wilde Lake watershed. As discussed earlier, preliminary investigation revealed that a very large number of combinations of sub-watersheds might be considered as possible collection areas. It was reported that, in order to limit the number of cases to be calculated, no more than six sub-watersheds would be considered a single collection area. Also many trivial and doubtful cases were neglected and it was found that 110 different collection areas could be defined which would permit 173 combinations for the entire watershed. The collection areas, which are virtually identical with the corresponding service areas, were determined by examination of topographic maps of the watershed and include every case where runoff from one or more sub-watersheds is apparently available at a single place. In each case, that point of collection was identified by the sub-watershed within which it was located and the collection area thus defined was assigned an index number. Tables 27, 28, and 29 list the collection areas identified, showing the sub-watersheds which make up each area, the treatment plant location, and the index number. Each separate table represents a single computer run and is assigned its own set of index numbers. The watershed was divided into three separate areas for analysis designated by Computer Runs A, B, and C, to simplify the computer runs and to make the results of the first analysis available at the earliest possible time, in the event that modifications to the method would be required. The limitation of six sub-watersheds in a single collection area permitted the total watershed to be so divided without any significant penalty in terms of total number of combinations presented for analysis.

COMBINATIONS OF COLLECTION AREAS

Having defined all the individual storm water storage, treatment, and reuse facilities possible within the Wilde Lake watershed, subject to the stated constraints, the combinations of these systems which would serve the entire watershed remain to be defined. Again, this may be done by inspection, and Tables 30, 31, and 32 list the combinations so identified, defined in terms of index number. Each index number, in turn, refers to the corresponding collection area for the same computer run. In other words, Combination Number 11 in Run B (Table 31) is made up of Index Numbers 2, 6, and 24, which are sub-watershed numbers 2, 6, and the combination of 1, 3, 11, and 12 (from Table 28). Each combination represents a portion of a complete system with storage, treatment, and reuse facilities for every area in that section of the watershed. The various methods of assembling the 173 combinations result in 186,300 ways of serving the entire watershed.

TABLE 27. COLLECTION AREAS - COMPUTER RUN A

<u>Index Number</u>	<u>Collection Area (Sub-watersheds)</u>						<u>Treatment Location</u>
1	4						4
2	5						5
3	6						6
4	7						7
5	8						8
6	14						14
7	4	5					4
8	4	14					14
9	5	14					14
10	6	8					6
11	6	7					7
12	7	8					7
13	7	14					14
14	4	5	14				14
15	4	7	14				14
16	5	7	14				14
17	6	7	14				14
18	7	8	14				7
19	6	7	8				7
20	4	5	7				7
21	5	6	7				7
22	5	7	8				7
23	4	6	7				7
24	4	7	8				7
25	4	5	7	14			14
26	4	6	7	14			14
27	5	6	7	14			14
28	4	7	8	14			14
29	5	7	8	14			14
30	6	7	8	14			14
31	4	5	6	7			7
32	4	5	7	8			7
33	4	6	7	8			7
34	5	6	7	8			7
35	4	5	6	8			6
36	4	5	6	7	14		14
37	4	5	7	8	14		14
38	4	6	7	8	14		14
39	5	6	7	8	14		14
40	4	5	6	7	8		7
41	4	5	6	7	8	14	14

TABLE 28. COLLECTION AREAS - COMPUTER RUN B

<u>Index Number</u>	<u>Collection Area (Sub-Watersheds)</u>						<u>Treatment Location</u>
1	1						1
2	2						2
3	3						3
4	11						11
5	12						12
6	13						13
7	1	2					2
8	1	3					3
9	2	3					3
10	3	11					11
11	3	12					12
12	11	12					11
13	11	13					13
14	12	13					13
15	1	2	3				3
16	1	3	11				11
17	1	3	12				12
18	2	3	11				11
19	2	3	12				12
20	3	11	12				11
21	3	11	13				13
22	11	12	13				13
23	1	2	3	11			11
24	1	3	11	12			11
25	1	3	11	13			13
26	2	3	11	12			11
27	2	3	11	13			13
28	3	11	12	13			13
29	1	2	3	11	12		11
30	1	2	3	11	13		13
31	1	3	11	12	13		13
32	2	3	11	12	13		13
33	1	2	3	11	12	13	13

TABLE 29. COLLECTION AREAS - COMPUTER RUN C

<u>Index Number</u>	<u>Collection Area (Sub-watersheds)</u>						<u>Treatment Location</u>
1	15						15
2	16						16
3	17						17
4	18						18
5	9						9
6	20						20
7	10						10
8	19						19
9	21						21
10	22						22
11	15	16					16
12	15	17					17
13	15	18					18
14	16	18					18
15	17	18					18
16	18	9					9
17	9	20					20
18	15	16	17				17
19	15	16	18				18
20	15	17	18				18
21	15	18	9				9
22	16	18	9				9
23	17	18	9				9
24	18	9	20				20
25	15	16	17	18			18
26	15	17	18	9			9
27	15	16	18	9			9
28	15	18	9	20			20
29	16	17	18	9			9
30	16	18	9	20			20
31	17	18	9	20			20
32	15	16	17	18	9		9
33	15	17	18	9	20		20
34	15	16	18	9	20		20
35	16	17	18	9	20		20
36	15	16	17	18	9	20	20

TABLE 30. COMBINATION OF AREAS - COMPUTER RUN A

Combination Number	Index Numbers of Collection Areas	Combination Number	Index Numbers of Collection Areas
1	41	35	19
2	40	36	19
3	39	37	19
4	38	38	18
5	37	39	18
6	36	40	17
7	35	41	17
8	35	42	16
9	34	43	16
10	34	44	15
11	33	45	15
12	33	46	14
13	32	47	14
14	31	48	14
15	30	49	14
16	30	50	13
17	29	51	13
18	28	52	13
19	27	53	13
20	26	54	12
21	25	55	12
22	25	56	12
23	24	57	12
24	24	58	11
25	23	59	11
26	23	60	11
27	22	61	11
28	22	62	10
29	21	63	10
30	21	64	10
31	20	65	10
32	20	66	9
33	19	67	8
34	19	68	7
		69	1

TABLE 32. COMBINATION OF AREAS - COMPUTER RUN C

Combination Number	Index Numbers of Collection Areas									
1	36	7	8	9	10					
2	35	1	7	8	9	10				
3	34	3	7	8	9	10				
4	33	2	7	8	9	10				
5	32	6	7	8	9	10				
6	31	11	7	8	9	10				
7	31	1	2	7	8	9	10			
8	30	12	7	8	9	10				
9	30	1	3	7	8	9	10			
10	29	1	6	7	8	9	10			
11	28	2	3	7	8	9	10			
12	27	3	6	7	8	9	10			
13	26	2	6	7	8	9	10			
14	25	17	7	8	9	10				
15	25	5	6	7	8	9	10			
16	24	18	7	8	9	10				
17	24	11	3	7	8	9	10			
18	24	12	2	7	8	9	10			
19	24	1	2	3	7	8	9	10		
20	23	11	6	7	8	9	10			
21	23	1	2	6	7	8	9	10		
22	22	12	6	7	8	9	10			
23	22	1	3	6	7	8	9	10		
24	21	2	3	6	7	8	9	10		
25	20	17	2	7	8	9	10			
26	20	2	5	6	7	8	9	10		
27	19	17	3	7	8	9	10			
28	19	3	5	6	7	8	9	10		
29	18	16	6	7	8	9	10			
30	18	17	4	7	8	9	10			
31	18	4	5	6	7	8	9	10		
32	17	11	15	7	8	9	10			
33	17	12	14	7	8	9	10			
34	17	13	2	3	7	8	9	10		
35	17	11	3	4	7	8	9	10		
36	17	12	2	4	7	8	9	10		
37	17	14	1	3	7	8	9	10		
38	17	15	1	2	7	8	9	10		
39	17	1	2	3	4	7	8	9	10	
40	16	11	3	6	7	8	9	10		
41	16	12	2	6	7	8	9	10		
42	16	1	2	3	6	7	8	9	10	
43	15	11	5	6	7	8	9	10		
44	15	1	2	5	6	7	8	9	10	
45	14	12	5	6	7	8	9	10		
46	14	1	3	5	6	7	8	9	10	
47	13	2	3	5	6	7	8	9	10	
48	12	2	4	5	6	7	8	9	10	
49	11	3	4	5	6	7	8	9	10	
50	1	2	3	4	5	6	7	8	9	10

Since each section of the complete watershed that was isolated for a computer run is, by the terms of the assumptions, independent of the others, each can be optimized separately and the optimum system for the watershed will be the summation of the individual optimum systems. This approach reduces by several orders of magnitude the number of cases which must be examined.

SECTION XI

SYSTEM MODEL OUTPUTS AND OPTIMIZATION

Section V discussed the water demands which are under consideration as possible areas of storm water reuse. It was determined that the four classes of demands could be served in eight different ways: the storm water system could be connected to the "AA, A, B, and C" demands, the "A, B, and C" demands, the "A and B," "A and C," or "B and C" demands, or simply be connected to the "A," "B," or "C" demands independently of the others. In each case, the distribution system and treatment system required are different from any of the other cases. Tables 27, 28, and 29 presented various collection areas that could be considered, each of them made up of one or more sub-watersheds, but permitting storm water to be collected, stored, and treated at a single central location. The first step in the optimization was, therefore, to compute total system costs for each feasible arrangement of facilities within each possible collection area.

TABLE OF NET BENEFITS

These results are displayed as a Table of Net Benefits, a sample of which is reproduced as Figure 34. The case numbers in the left-hand column can be observed to correspond to the index numbers of Table 27. Each line represents a particular collection area, with the sub-watersheds and treatment plant location listed in successive columns. The entries under the eight water service level columns correspond to the net system benefit for the facilities which would serve the demands indicated in the collection area listed. The final column reflects the maximum net benefit appearing on that line and identifies the optimum configuration for that collection area alone. Negative net benefits are identical with net system costs. All values are in dollars/day and are computed as follows:

$$B_n = B_g - C_{jo} - C_p - C_t - C_h - C_{ko} - C_d \quad (32)$$

where:

- B_n = net system benefits (dollars/day)
- B_g = gross system benefit, computed as discussed (see Section XI) (dollars/day)
- C_{jo} = sub-optimized system cost of storm water storage from Equation 33 (dollars/day)
- C_p = system cost of pretreatment from Equation 34 (dollars/day)

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**** TABLE OF NET BENEFITS ****									
CASE NUMBER	WATERSHED NUMBER	PLANT LOCATION	- WATER QUALITY -					AA,A,B,C	MAX
			A	B	C	A,B	A,C	A,B,C	
1	4	4	-9.18	-7.54	-5.78	-8.60	-8.60	-8.60	-5.78
2	5	5	-10.74	-10.09	-7.12	-9.80	-9.17	-9.17	-6.52
3	6	6	-41.24	-27.41	-27.72	-41.24	-41.24	-41.24	-4.16
4	7	7	-21.72	-16.96	-12.08	-21.72	-21.72	-21.72	-4.57
5	8	8	-10.76	-9.07	-6.03	-8.91	-8.88	-8.88	-5.26
6	14	14	-19.77	-15.59	-13.47	-16.43	-16.43	-16.43	-4.48
7	4	4	-15.56	-12.84	-10.71	-13.43	-13.43	-13.43	-4.48
8	14	14	-24.93	-18.65	-17.36	-21.06	-21.06	-21.06	-3.55
9	5	5	-25.95	-20.64	-18.14	-21.47	-21.11	-21.11	-2.93
10	6	6	-45.92	-31.93	-27.75	-45.92	-45.92	-45.92	-1.39
11	7	7	-58.38	-39.64	-37.85	-58.38	-58.38	-58.38	-0.14
12	8	8	-26.75	-21.58	-16.35	-26.75	-26.75	-26.75	-2.40
13	14	14	-35.28	-28.06	-23.79	-33.97	-33.97	-33.97	-1.00
14	4	4	-31.15	-23.69	-22.09	-25.52	-25.52	-25.52	-1.54
15	5	5	-40.25	-31.16	-27.75	-38.31	-38.31	-38.31	-0.53
16	6	6	-41.19	-33.13	-28.55	-38.17	-38.17	-38.17	1.51
17	7	7	-69.22	-50.75	-43.51	-69.22	-69.22	-69.22	5.09
18	8	8	-41.47	-32.52	-27.93	-38.13	-38.13	-38.13	2.71
19	14	14	-62.49	-44.21	-37.99	-62.49	-62.49	-62.49	3.52
20	4	4	-31.02	-25.16	-20.85	-31.02	-31.02	-31.02	-1.95
21	5	5	-62.58	-44.71	-39.16	-62.58	-62.58	-62.58	2.37
22	6	6	-32.32	-26.62	-21.07	-30.95	-30.95	-30.95	0.11
23	7	7	-62.71	-42.75	-40.31	-62.71	-62.71	-62.71	1.39
24	8	8	-31.38	-24.63	-20.29	-31.09	-31.09	-31.09	-0.87
25	14	14	-46.15	-36.24	-32.55	-42.59	-42.59	-42.59	3.09
26	4	4	-73.07	-53.86	-46.11	-73.07	-73.07	-73.07	6.80
27	5	5	-72.63	-55.82	-46.90	-72.63	-72.63	-72.63	7.89
28	6	6	-46.54	-35.74	-32.03	-42.15	-42.15	-42.15	4.29
29	7	7	-47.48	-37.70	-32.83	-41.71	-41.71	-41.71	5.37
30	8	8	-72.39	-55.32	-46.35	-72.39	-72.39	-72.39	9.09
31	14	14	-66.58	-47.82	-41.57	-66.58	-66.58	-66.58	4.02
32	4	4	-37.28	-29.69	-25.05	-35.28	-35.28	-35.28	1.65
33	5	5	-65.90	-47.32	-40.38	-66.34	-66.34	-66.34	5.22
34	6	6	-54.44	-40.11	-39.44	-54.44	-54.44	-54.44	6.31
35	7	7	-76.47	-58.93	-50.85	-76.47	-76.47	-76.47	2.66
36	8	8	-52.18	-40.81	-36.83	-45.56	-45.56	-45.56	9.61
37	14	14	-76.20	-58.77	-50.30	-76.20	-76.20	-76.20	7.08
38	4	4	-75.70	-60.44	-51.06	-75.70	-75.70	-75.70	10.84
39	5	5	-69.75	-52.39	-43.39	-69.75	-69.75	-69.75	11.99
40	6	6	-79.40	-64.12	-54.88	-79.40	-79.40	-79.40	8.02
41	7	7							13.85

Figure 34. Table of Net Benefits - Computer Run A

C_t	=	system cost of final treatment from Equation 35 (dollars/day)
C_h	=	system cost of pumping from Equation 36 (dollars/day)
C_{ko}	=	sub-optimized system cost of treated water storage from Equation 37 (dollars/day)
C_d	=	system cost of distribution from Equation 38 (dollars/day)

TABLES OF CAPITAL COSTS AND DAILY COSTS

Figures 35 and 36 show computer listings of capital costs and daily operating and maintenance costs for each of the collection area/reuse level alternatives considered in one of the computer runs. The Table of Capital Cost indicates the total construction cost, in dollars, for each alternative. Case Number 11, for example, is a collection area composed of sub-watersheds Numbers 6 and 7 and for the reuse level "C" (only Class "C" uses considered) has a capital cost of 0.03958E 06, read as 0.03958×10^6 , or \$39,580. This includes all costs associated with the construction of a storm water storage facility, a pretreatment unit, pumping facilities, treated water storage, and a "C" level distribution system. No final treatment was provided, since only Class "C" water was required. The next entry on the same line, for reuse level "A, B," indicated a capital cost of 0.4658E 06 or \$46,580 and includes a Class "A" treatment plant as well as the additional cost of the "A, B" distribution system. Pumping and treated water storage facilities may be sized somewhat differently due to increased total demand.

The Table of Daily Cost, Figure 36, follows the same format as Figure 35, except that entries are for total operating and maintenance cost, expressed in dollars/day. The alternative used in the example above, Case Number 11 at reuse level "C," can be found to have an operating and maintenance cost of \$19.70 per day. The "A, B" reuse level for the same case has an operating and maintenance cost of \$30.20 per day, largely due to the additional cost of operating the Class "A" treatment plant. These costs include silt removal, maintenance costs of storm water and treated water storage, where applicable, operating and maintenance costs of pretreatment, final treatment, pumping, and distribution facilities. The total system cost of each alternative in dollars/day is the sum of the daily costs from Figure 36 and the capital costs from Figure 35 multiplied by the capital recovery factor and divided by 365.

TABLE OF CAPITAL COST **

LASE NUMBER	PLANT LOCATION	WATERSHED NUMBER	A	B	C	A+B	A-C	A+B-C	B-C	A+A+B+C
1	4	0	0.7069E	050.6113E	050.5139E	050.7100E	050.7100E	050.7100E	050.5982E	050.5752E 05
2	5	0	0.8307E	050.6491E	050.6231E	050.8391E	050.8391E	050.8391E	050.8081E	050.6405E 05
3	6	0	0.3145E	060.2060E	060.2642E	060.3145E	060.3145E	060.3145E	060.2835E	060.7175E 05
4	7	0	0.1800E	060.1233E	060.1454E	060.1800E	060.1800E	060.1800E	060.1644E	060.6493E 05
5	8	0	0.8976E	050.6836E	050.6609E	050.9073E	050.9073E	050.9073E	050.8495E	050.6257E 05
6	14	0	0.1505E	060.1227E	060.1246E	060.1528E	060.1528E	060.1528E	060.1446E	060.6894E 05
7	9	0	0.1235E	060.9704E	050.9845E	050.1248E	060.1248E	060.1248E	060.1175E	060.6644E 05
8	14	0	0.1933E	060.1527E	060.1629E	060.1963E	060.1963E	060.1963E	060.1839E	060.7394E 05
9	14	0	0.2013E	060.1567E	060.1700E	060.2053E	060.2053E	060.2053E	060.1910E	060.7686E 05
10	6	0	0.3773E	060.2427E	060.3161E	060.3773E	060.3773E	060.3773E	060.3382E	060.8194E 05
11	7	0	0.4628E	060.2968E	060.3958E	060.4628E	060.4628E	060.4628E	060.4161E	060.8293E 05
12	7	0	0.2441E	060.1607E	060.1992E	060.2441E	060.2441E	060.2441E	060.2199E	060.7566E 05
13	14	0	0.3021E	060.2148E	060.2578E	060.3060E	060.3060E	060.3060E	060.2792E	060.8203E 05
14	14	0	0.2449E	060.1867E	060.2086E	060.2490E	060.2490E	060.2490E	060.2305E	060.8181E 05
15	14	0	0.3431E	060.2451E	060.2964E	060.3494E	060.3494E	060.3494E	060.3184E	060.8698E 05
16	14	0	0.3567E	060.2489E	060.3035E	060.3587E	060.3587E	060.3587E	060.3261E	060.8989E 05
17	14	0	0.5925E	060.3880E	060.5086E	060.5925E	060.5925E	060.5925E	060.5302E	060.1031E 06
18	7	0	0.3663E	060.2510E	060.3102E	060.3666E	060.3666E	060.3666E	060.3332E	060.9143E 05
19	7	0	0.5294E	060.3335E	060.4497E	060.5294E	060.5294E	060.5294E	060.4708E	060.9397E 05
20	7	0	0.2769E	060.1881E	060.2304E	060.2769E	060.2769E	060.2769E	060.2513E	060.7852E 05
21	7	0	0.5185E	060.3306E	060.4421E	060.5185E	060.5185E	060.5185E	060.4630E	060.9080E 05
22	7	0	0.2959E	060.1947E	060.2449E	060.2968E	060.2968E	060.2968E	060.2668E	060.8372E 05
23	7	0	0.5092E	060.3268E	060.4347E	060.5092E	060.5092E	060.5092E	060.4554E	060.8788E 05
24	7	0	0.2873E	060.1907E	060.2378E	060.2875E	060.2875E	060.2875E	060.2591E	060.8081E 05
25	14	0	0.3997E	060.2793E	060.3421E	060.4024E	060.4024E	060.4024E	060.3654E	060.9551E 05
26	14	0	0.6370E	060.4163E	060.5474E	060.6370E	060.6370E	060.6370E	060.5695E	060.1109E 06
27	14	0	0.6470E	060.4222E	060.5545E	060.6470E	060.6470E	060.6470E	060.5772E	060.1156E 06
28	14	0	0.4101E	060.2822E	060.3496E	060.4137E	060.4137E	060.4137E	060.3732E	060.9962E 05
29	14	0	0.4186E	060.2860E	060.3566E	060.4237E	060.4237E	060.4237E	060.3809E	060.1043E 06
30	14	0	0.6583E	060.4250E	060.5619E	060.6583E	060.6583E	060.6583E	060.5851E	060.1197E 06
31	7	0	0.5626E	060.3610E	060.4810E	060.5626E	060.5626E	060.5626E	060.5023E	060.9769E 05
32	7	0	0.3896E	060.2248E	060.2835E	060.3403E	060.3403E	060.3403E	060.3060E	060.8867E 05
33	7	0	0.5740E	060.3639E	060.4885E	060.5740E	060.5740E	060.5740E	060.5102E	060.1018E 06
34	7	0	0.5840E	060.3677E	060.4959E	060.5840E	060.5840E	060.5840E	060.5178E	060.1065E 06
35	6	0	0.4741E	060.3073E	060.4030E	060.4741E	060.4741E	060.4741E	060.4244E	060.9487E 05
36	14	0	0.6914E	060.4525E	060.5930E	060.6914E	060.6914E	060.6914E	060.6164E	060.1233E 06
37	14	0	0.4622E	060.3164E	060.3952E	060.4682E	060.4682E	060.4682E	060.4202E	060.1121E 06
38	14	0	0.7025E	060.4554E	060.6004E	060.7025E	060.7025E	060.7025E	060.6241E	060.1272E 06
39	14	0	0.7121E	060.4590E	060.6073E	060.7121E	060.7121E	060.7121E	060.6314E	060.1315E 06
40	7	0	0.6285E	060.3981E	060.5345E	060.6285E	060.6285E	060.6285E	060.5572E	060.1143E 06
41	14	0	0.7556E	060.4887E	060.6449E	060.7556E	060.7556E	060.7556E	060.6696E	060.1382E 06

Figure 35. Table of Capital Cost - Computer Run A

TABLE OF DAILY COST

CASE NUMBER	WATERSHED NUMBER			PLANT LOCATION	MATER QUALITY				A.A.A.B.C		
	A	B	C		A.B	A.C	A.B.C	B.C	A.A.A.B.C		
1	4	0	0	0	0	0	0	0	4.27	3.04	4.21
2	5	0	0	0	0	0	0	0	6.62	3.82	6.62
3	6	0	0	0	0	0	0	0	17.84	12.99	15.57
4	7	0	0	0	0	0	0	0	12.81	9.16	12.17
5	8	0	0	0	0	0	0	0	9.05	5.79	9.47
6	14	0	0	0	0	0	0	0	13.63	9.57	13.72
7	4	0	0	0	0	0	0	0	10.88	7.24	11.02
8	4	14	0	0	0	0	0	0	17.88	12.69	17.91
9	5	14	0	0	0	0	0	0	20.23	13.37	20.52
10	6	8	0	0	0	0	0	0	26.62	20.24	24.53
11	6	7	0	0	0	0	0	0	23.65	23.65	26.86
12	8	7	0	0	0	0	0	0	21.83	15.46	21.61
13	7	14	0	0	0	0	0	0	26.11	20.01	25.26
14	4	5	14	0	0	0	0	0	24.26	17.32	24.27
15	4	7	14	0	0	0	0	0	30.07	24.21	28.87
16	5	7	14	0	0	0	0	0	32.24	26.54	31.11
17	6	7	14	0	0	0	0	0	41.99	34.68	37.86
18	6	7	14	0	0	0	0	0	41.99	34.68	37.86
19	6	7	14	0	0	0	0	0	34.48	28.92	33.39
20	4	5	7	0	0	0	0	0	38.36	32.15	34.81
21	6	7	5	0	0	0	0	0	23.56	17.27	22.94
22	6	7	5	0	0	0	0	0	36.32	30.38	32.71
23	6	7	4	0	0	0	0	0	27.96	21.99	27.46
24	6	7	4	0	0	0	0	0	34.16	28.05	30.87
25	6	7	4	0	0	0	0	0	25.79	19.66	25.22
26	4	5	7	14	0	0	0	0	36.05	30.27	34.59
27	4	6	7	14	0	0	0	0	45.30	36.89	40.49
28	4	6	7	14	0	0	0	0	47.07	37.98	42.76
29	4	6	7	14	0	0	0	0	37.88	31.39	36.49
30	4	6	7	14	0	0	0	0	39.65	32.49	38.37
31	4	5	6	7	0	0	0	0	48.90	39.11	44.67
32	4	5	6	7	0	0	0	0	39.85	33.23	35.92
33	4	6	7	8	0	0	0	0	31.92	26.19	31.07
34	4	6	7	8	0	0	0	0	41.68	34.36	37.83
35	4	5	6	8	0	0	0	0	43.44	35.45	39.70
36	4	5	6	7	14	0	0	0	36.68	30.89	33.97
37	4	5	6	7	14	0	0	0	50.39	40.19	45.79
38	4	6	7	8	14	0	0	0	42.96	34.69	41.39
39	4	6	7	8	14	0	0	0	52.21	41.31	47.69
40	4	5	6	7	8	14	0	0	53.98	42.40	49.57
41	4	5	6	7	8	14	0	0	46.76	37.66	42.73
									57.29	44.61	52.59

Figure 36. Table of Daily Cost - Computer Run A

TABLE OF BENEFITS

The gross system benefits for the alternatives discussed in the preceding sections were calculated and displayed on a Table of Benefits, shown as Figure 37. Benefits were taken as the marginal cost from the alternate source of the water used. In this case, benefits were the marginal cost, to the consumer, of an equal volume of water purchased from the Howard County Metropolitan Commission. Under the rate structure in effect in July 1968, that cost is \$0.45 per 1000 gallons. The use of this very elementary measure of benefits overlooks the effect of such use on the public water supply system. Many of the costs associated with providing the public water supply to the demand area under study would not be reduced by the introduction of the supplementary source. The distribution system, the cost of billing and collection, the overhead structure of the Metropolitan Commission - all of these would be essentially unchanged. Since all of the water distributed by Howard County is purchased from Baltimore City and the Washington Suburban Sanitary Commission, the most clear-cut reduction in true costs would be the reduction in the quantity of water purchased, which accounts for less than half of the total selling price of water. Other costs would be reduced in a less easily determined manner, such as booster pumping costs, storage costs, transmission main costs, etc. To properly evaluate the total impact of the study system on the public supply system, a detailed study of the financial structure of the Metropolitan Commission would be required, a task outside the scope of this work. Since the Wilde Lake area constitutes a small portion of the county water distribution area, it was considered that the elimination of a portion of the demand in this watershed would not have a significant overall effect on the cost of supplying water to the county. For this reason, the full price of water was taken as a benefit, although in a proposed large-scale application of the local reuse concept, this assumption would require careful review in light of local circumstances. Ideally, the economic benefit due to water supply should be taken as the marginal reduction in the total long-term costs of the public water system, due to the reduction in demand on that system.

At the time the computer runs were performed, the marginal cost of water in Howard County had recently been revised to \$0.50 per 1000 gallons. Shortly after the runs were completed, however, the rate was reduced to \$0.45 per 1000 gallons. Accordingly, Figure 37 shows benefits calculated on the basis of the higher figure, and subsequent discussion of net benefits in this chapter will be in terms of \$0.50 per 1000 gallons marginal cost of water. Prior to the definition of the conceptual optimum systems described in Sections XII-XV, however, and prior to the system evaluation reported in Section XVII, the benefit rate was revised to \$0.45 per 1000 gallons and economic comparisons made on that basis. Since the benefit is, in this case, completely linear and independent of other assumptions, manipulating the rate has no effect on system optima. The individual benefits listed on Figure 37 are calculated as the product of the treated water demand in gallons per day and marginal cost of water to consumer from public system in dollars per gallon. The net system benefits can also be calculated as the difference between the gross system benefits on Figure 37 and the total system costs (Capital Cost from Figure 35 ÷ 365 x Capital Recovery factor + Daily Cost from Figure 36).

TABLE OF BENEFITS

CASE NUMBER	WATERSHED NUMBER	PLANT LOCATION	WATER QUALITY				A.A.B.C
			A	B	C	A.B	
1	4	4	4.59	3.40	3.74	5.85	5.85
2	5	5	6.07	2.04	4.70	8.11	9.49
3	6	6	21.71	10.49	21.71	21.71	21.71
4	7	7	16.91	5.91	16.91	16.91	16.91
5	8	8	9.11	3.99	7.53	13.10	13.10
6	14	14	11.75	7.95	10.84	19.12	19.12
7	14	14	10.66	5.44	8.44	15.34	15.34
8	14	14	16.34	11.35	14.58	24.97	24.97
9	14	14	17.82	9.99	15.55	27.82	28.61
10	6	6	34.89	14.48	34.89	34.89	34.89
11	6	6	38.61	16.41	38.61	38.61	38.61
12	7	7	30.09	9.90	24.29	30.09	30.09
13	7	7	33.40	13.87	27.61	36.03	36.03
14	14	14	22.41	13.39	19.29	34.46	34.46
15	14	14	37.99	17.27	31.35	41.88	41.88
16	5	5	39.48	15.91	32.31	45.52	45.52
17	6	6	57.73	24.36	57.73	57.73	57.73
18	7	7	42.52	17.66	35.14	49.21	49.21
19	7	7	51.80	20.40	51.80	51.80	51.80
20	7	7	32.25	11.35	25.21	32.25	32.25
21	7	7	48.10	18.45	48.10	48.10	48.10
22	7	7	30.84	11.94	29.00	39.58	39.58
23	7	7	44.47	19.81	44.47	44.47	44.47
24	7	7	35.36	13.30	28.03	35.94	35.94
25	4	4	63.59	27.76	63.59	63.59	63.59
26	4	4	67.22	26.40	64.27	67.22	67.22
27	4	4	47.11	21.26	38.88	55.06	55.06
28	4	4	48.59	19.90	39.84	58.70	58.70
29	4	4	70.92	28.35	67.09	70.92	70.92
30	4	4	53.96	21.84	53.96	53.96	53.96
31	4	4	41.43	15.34	32.74	45.43	45.43
32	4	4	57.65	23.80	57.65	57.65	57.65
33	4	4	61.29	22.44	60.95	61.29	61.29
34	4	4	50.23	19.92	47.93	50.23	50.23
35	4	4	73.08	29.80	68.01	73.08	73.08
36	4	4	53.18	23.30	43.58	64.55	64.55
37	4	4	76.77	31.75	70.83	76.77	76.77
38	4	4	80.41	30.39	71.80	80.41	80.41
39	4	4	67.14	25.83	64.69	67.14	67.14
40	4	4	86.26	33.79	75.54	86.26	86.26
41	4	4					

Figure 37. Table of Benefits - Computer Run A

TABLE OF NET BENEFITS FOR COMBINATIONS OF WATERSHEDS

Tables 30, 31, and 32, described earlier, defined the various combinations of collection areas which would satisfy the entire section of the watershed under consideration. The entire watershed was analyzed in three sections, each the subject of a single computer run. In order to compare and rank the various alternative systems which would serve the section under study, it is necessary, for each combination of collection areas, to add the net benefits associated with the most attractive reuse levels for each of the component collection areas, obtaining the total net benefit for the combination. This is done by adding the net benefits appearing in the final column of the Table of Net Benefits (Figure 34) for the collection areas which make up a given combination. When this is done for each possible combination, the results can be ranked and the optimum system for that section identified.

In order to obtain as much information about alternative strategies as possible, a five-level output was obtained. This output defined optimum systems under each of the following assumptions:

1. All levels of reuse possible
2. Only reuse up to water quality "A" possible
3. Only reuse up to water quality "B" possible
4. Only reuse up to water quality "C" possible
5. No reuse possible

This output for computer Run A is displayed as Figures 38 and 39. Figure 38 lists total net benefits for all possible combinations under the first four assumptions noted above. Column "AA" reflects combinations of optimum subsystems when all levels of reuse are possible. It should be noted that the optimum subsystems might be composed of any assortment of reuses. The only requirement is that each collection area be served by the system having the maximum net benefit for that area. These subsystems are those identified by the MAX column of Figure 34. Column "A" follows the same pattern, except that all subsystems involving the use of "AA" water are excluded from the analysis. Column "B" excludes all "AA" and "A" uses and Column "C" excludes all but "C." The assumption of no reuse required a separate computer run, due to the limitation of the system to storm water storage and pretreatment facilities. Since no benefits, as defined in the economic analysis, derive from this application, the net benefits reported are identical with system cost. Figure 39 lists the totals for the various combinations. This assumption describes a system which retains only water pollution control as its purpose and permits direct comparison with the alternative method of water pollution control described in later chapters.

**** TABLE OF NET BENEFITS FOR COMBINATIONS OF WATERSHEDS ****

COMBINATION NUMBER	PLANT COMBINATIONS, PERMISSIBLE MAXIMUM QUALITY										AA	A	B	C
1	41	0	0	0	0	0	0	0	0	0	13.85	-54.38	-54.38	-54.86
2	40	6	0	0	0	0	0	0	0	0	3.54	-54.90	-54.90	-56.86
3	39	1	0	0	0	0	0	0	0	0	6.20	-56.84	-56.84	-56.84
4	38	2	0	0	0	0	0	0	0	0	4.33	-57.42	-57.42	-57.42
5	37	3	0	0	0	0	0	0	0	0	2.93	-57.82	-57.82	-64.55
6	36	5	0	0	0	0	0	0	0	0	4.35	-56.88	-56.88	-56.88
7	35	13	0	0	0	0	0	0	0	0	1.66	-57.00	-57.00	-57.00
8	35	4	6	0	0	0	0	0	0	0	-6.40	-56.79	-56.79	-58.75
9	34	8	0	0	0	0	0	0	0	0	2.77	-53.54	-53.54	-56.80
10	34	1	6	0	0	0	0	0	0	0	-3.96	-56.72	-56.72	-58.69
11	33	9	0	0	0	0	0	0	0	0	2.29	-55.60	-55.60	-58.53
12	33	2	6	0	0	0	0	0	0	0	-5.78	-59.00	-59.00	-60.77
13	32	3	6	0	0	0	0	0	0	0	-6.99	-63.55	-63.55	-66.24
14	31	5	6	0	0	0	0	0	0	0	-5.72	-59.11	-59.11	-61.08
15	30	7	0	0	0	0	0	0	0	0	3.88	-56.56	-56.56	-57.06
16	30	1	2	0	0	0	0	0	0	0	-3.21	-59.25	-59.25	-59.25
17	29	1	3	0	0	0	0	0	0	0	-4.57	-61.61	-61.61	-66.34
18	28	2	3	0	0	0	0	0	0	0	-6.39	-64.39	-64.39	-66.88
19	27	1	5	0	0	0	0	0	0	0	-3.15	-58.71	-58.71	-58.71
20	26	2	5	0	0	0	0	0	0	0	-4.97	-59.26	-59.26	-59.26
21	25	10	0	0	0	0	0	0	0	0	1.69	-59.05	-59.05	-60.30
22	25	3	5	0	0	0	0	0	0	0	-6.33	-64.74	-64.74	-66.30
23	24	9	3	0	0	0	0	0	0	0	-7.95	-62.92	-62.92	-66.15
24	24	2	3	6	0	0	0	0	0	0	-16.02	-66.32	-66.32	-68.60
25	23	9	5	0	0	0	0	0	0	0	-6.80	-61.56	-61.56	-64.48
26	23	2	5	6	0	0	0	0	0	0	-14.86	-64.96	-64.96	-66.93
27	22	4	3	0	0	0	0	0	0	0	-7.59	-62.18	-62.18	-64.15
28	22	1	3	6	0	0	0	0	0	0	-14.31	-65.36	-65.36	-68.05
29	21	8	5	0	0	0	0	0	0	0	-6.43	-59.29	-59.29	-62.55
30	21	1	5	6	0	0	0	0	0	0	-13.15	-62.48	-62.48	-64.45
31	20	10	6	0	0	0	0	0	0	0	-7.83	-60.11	-60.11	-62.07
32	20	3	5	6	0	0	0	0	0	0	-15.85	-65.80	-65.80	-68.07
33	19	14	0	0	0	0	0	0	0	0	1.98	-55.69	-55.69	-60.08
34	19	7	6	0	0	0	0	0	0	0	-6.17	-59.70	-59.70	-62.17
35	19	8	2	0	0	0	0	0	0	0	-6.55	-59.21	-59.21	-62.47
36	19	9	1	0	0	0	0	0	0	0	-5.20	-58.99	-58.99	-61.92
37	19	1	2	6	0	0	0	0	0	0	-13.27	-62.40	-62.40	-64.34
38	18	7	3	0	0	0	0	0	0	0	-6.65	-65.11	-65.11	-66.36
39	18	1	2	3	0	0	0	0	0	0	-13.75	-67.80	-67.80	-68.55
40	17	7	5	0	0	0	0	0	0	0	-5.37	-59.75	-59.75	-60.25
41	17	1	2	5	0	0	0	0	0	0	-12.46	-62.45	-62.45	-62.45
42	16	10	1	0	0	0	0	0	0	0	-5.66	-61.33	-61.33	-62.09
43	16	1	3	5	0	0	0	0	0	0	-13.68	-67.02	-67.02	-68.05
44	15	10	2	0	0	0	0	0	0	0	-7.37	-62.63	-62.63	-62.63
45	15	2	3	5	0	0	0	0	0	0	-15.40	-68.32	-68.32	-68.63
46	14	11	5	0	0	0	0	0	0	0	-6.94	-61.58	-61.58	-65.97
47	14	12	3	0	0	0	0	0	0	0	-8.09	-61.46	-61.46	-66.16
48	14	10	4	0	0	0	0	0	0	0	-7.51	-57.54	-57.54	-61.92
49	14	3	4	5	0	0	0	0	0	0	-15.53	-63.23	-63.23	-67.92
50	13	7	10	0	0	0	0	0	0	0	-7.60	-61.76	-61.76	-62.26
51	13	7	3	5	0	0	0	0	0	0	-15.62	-67.45	-67.45	-68.26
52	13	10	1	2	0	0	0	0	0	0	-14.69	-64.45	-64.45	-64.45
53	13	1	2	3	5	0	0	0	0	0	-22.71	-70.14	-70.14	-70.45
54	12	7	3	6	0	0	0	0	0	0	-16.24	-65.47	-65.47	-68.25
55	12	8	2	3	0	0	0	0	0	0	-16.61	-64.98	-64.98	-68.55
56	12	9	1	3	0	0	0	0	0	0	-15.27	-64.76	-64.76	-68.00
57	12	1	2	3	6	0	0	0	0	0	-23.34	-68.17	-68.17	-70.44
58	11	7	5	6	0	0	0	0	0	0	-15.09	-65.59	-65.59	-68.06
59	11	8	2	5	0	0	0	0	0	0	-15.46	-65.10	-65.10	-68.36
60	11	9	1	5	0	0	0	0	0	0	-14.11	-64.88	-64.88	-67.81
61	11	1	2	5	6	0	0	0	0	0	-22.18	-68.29	-68.29	-70.25
62	10	7	4	6	0	0	0	0	0	0	-15.66	-61.55	-61.55	-64.32
63	10	8	2	4	0	0	0	0	0	0	-16.03	-61.06	-61.06	-63.76
64	10	9	1	4	0	0	0	0	0	0	-14.68	-60.84	-60.84	-63.21
65	10	1	2	4	6	0	0	0	0	0	-22.75	-64.24	-64.24	-69.76
66	9	1	3	4	5	0	0	0	0	0	-22.70	-66.53	-66.53	-70.32
67	8	2	3	4	5	0	0	0	0	0	-24.05	-66.75	-66.75	-70.02
68	7	3	4	5	6	0	0	0	0	0	-23.68	-67.24	-67.24	-72.21
69	1	2	3	4	5	6	0	0	0	0	-30.77	-69.93	-69.93	-72.21

Figure 38. Table of Net Benefits of Combinations of Watersheds, Reuse - Computer Run A

*** TABLE OF NET BENEFITS FOR COMBINATIONS OF WATERSHEDS

COMBINATION PLANT COMBINATIONS, PERMISSIBLE MAXIMUM QUALITY

NUMBER		NO REUSE
1	41 1 1 1 1 1 1 1 1 1 1	-15.99
2	49 1 1 1 1 1 1 1 1 1 1	-17.14
3	39 1 1 1 1 1 1 1 1 1 1	-16.80
4	38 2 1 1 1 1 1 1 1 1 1	-16.92
5	37 3 1 1 1 1 1 1 1 1 1	-17.14
6	36 5 1 1 1 1 1 1 1 1 1	-16.96
7	35 13 1 1 1 1 1 1 1 1 1	-17.12
8	35 4 1 1 1 1 1 1 1 1 1	-16.98
9	34 8 1 1 1 1 1 1 1 1 1	-17.14
10	34 1 1 1 1 1 1 1 1 1 1	-17.86
11	33 9 1 1 1 1 1 1 1 1 1	-17.13
12	33 2 6 1 1 1 1 1 1 1 1	-17.95
13	32 3 6 1 1 1 1 1 1 1 1	-18.17
14	31 5 6 1 1 1 1 1 1 1 1	-18.60
15	30 7 2 1 1 1 1 1 1 1 1	-17.69
16	30 1 2 1 1 1 1 1 1 1 1	-17.63
17	29 1 3 1 1 1 1 1 1 1 1	-17.85
18	28 2 3 1 1 1 1 1 1 1 1	-17.95
19	27 1 5 1 1 1 1 1 1 1 1	-17.67
20	26 2 5 1 1 1 1 1 1 1 1	-17.77
21	25 10 1 1 1 1 1 1 1 1 1	-17.12
22	25 3 5 1 1 1 1 1 1 1 1	-17.99
23	24 9 3 1 1 1 1 1 1 1 1	-18.16
24	24 2 3 6 1 1 1 1 1 1 1	-18.98
25	23 9 5 1 1 1 1 1 1 1 1	-17.98
26	23 2 5 6 1 1 1 1 1 1 1	-18.80
27	22 8 3 1 1 1 1 1 1 1 1	-18.16
28	22 1 3 6 1 1 1 1 1 1 1	-18.98
29	21 8 5 1 1 1 1 1 1 1 1	-17.99
30	21 1 5 6 1 1 1 1 1 1 1	-18.71
31	20 10 6 1 1 1 1 1 1 1 1	-18.16
32	20 3 5 6 1 1 1 1 1 1 1	-19.63
33	19 14 1 1 1 1 1 1 1 1 1	-17.12
34	19 7 6 1 1 1 1 1 1 1 1	-18.12
35	19 8 2 1 1 1 1 1 1 1 1	-17.94
36	19 9 1 1 1 1 1 1 1 1 1	-17.84
37	19 1 2 5 1 1 1 1 1 1 1	-18.66
38	18 7 3 1 1 1 1 1 1 1 1	-19.11
39	18 1 2 3 1 1 1 1 1 1 1	-18.66
40	17 7 5 1 1 1 1 1 1 1 1	-17.94
41	17 1 2 5 1 1 1 1 1 1 1	-18.48
42	16 10 1 1 1 1 1 1 1 1 1	-17.83
43	16 1 3 5 1 1 1 1 1 1 1	-18.70
44	15 10 2 1 1 1 1 1 1 1 1	-17.93
45	15 2 3 5 1 1 1 1 1 1 1	-18.80
46	14 11 5 1 1 1 1 1 1 1 1	-17.97
47	14 12 3 1 1 1 1 1 1 1 1	-18.16
48	14 10 4 1 1 1 1 1 1 1 1	-18.06
49	14 3 4 5 1 1 1 1 1 1 1	-18.93
50	13 7 10 1 1 1 1 1 1 1 1	-18.16
51	13 7 3 5 1 1 1 1 1 1 1	-18.96
52	13 10 1 2 1 1 1 1 1 1 1	-18.64
53	13 1 2 3 5 1 1 1 1 1 1	-19.51
54	12 7 3 6 1 1 1 1 1 1 1	-19.15
55	12 8 2 3 1 1 1 1 1 1 1	-18.97
56	12 9 1 3 1 1 1 1 1 1 1	-18.87
57	12 1 2 3 6 1 1 1 1 1 1	-19.69
58	11 7 5 6 1 1 1 1 1 1 1	-18.97
59	11 8 2 5 1 1 1 1 1 1 1	-18.79
60	11 9 1 5 1 1 1 1 1 1 1	-18.69
61	11 1 2 5 6 1 1 1 1 1 1	-19.51
62	10 7 4 6 1 1 1 1 1 1 1	-19.05
63	10 8 2 4 1 1 1 1 1 1 1	-18.88
64	10 9 1 4 1 1 1 1 1 1 1	-18.78
65	10 1 2 4 6 1 1 1 1 1 1	-19.60
66	9 1 3 4 5 1 1 1 1 1 1	-19.65
67	8 2 3 4 5 1 1 1 1 1 1	-19.74
68	7 3 4 5 6 1 1 1 1 1 1	-19.92
69	1 2 3 4 5 6 1 1 1 1 1	-20.46

Figure 39. Table of Net Benefits of Combinations of Watersheds, No Reuse - Computer Run A

OPTIMIZATION OF SYTEM

The discussion of sizing criteria for multiple purpose reservoirs which appears in Section VII included reference to various physical and aesthetic constraints which exist at reservoir sites. Since the application of these criteria is a function of the size of reservoir required at a given site as well as the type of reservoir selected, application of these constraints was not attempted within the systems analysis. In order to define the final optimum system, however, tentative optima must be tested for the physical and aesthetic constraints not included in the computer analysis. Many of these factors are determined by the exercise of judgement on the part of the engineer, the owner, and the owner's representatives, including architects and land planners. For these reasons and others, they are not considered susceptible to quantitative analysis. The method used to test for these constraints was as follows:

1. The total net benefits for a particular section of the watershed and particular level of output were placed in order of decreasing net benefit.
2. The highest-ranking system was defined in terms of reservoir types, sizes, and locations.
3. Each location was examined physically and via topographic and land use maps for feasibility of installation of the required facility.
4. If any of the reservoirs in a given combination appeared not feasible due to insufficient usable space, appearance, impact on the neighborhood, etc., that combination was discarded and the next-highest ranking one selected.
5. The process was repeated until the optimum feasible system was located for the section and output level.
6. The same method was applied to each output level and to each watershed section until all final optimum systems were defined.

These final systems are described and discussed in Sections XII, XIII, XIV, and XV. Information and guidance were obtained from the planning and engineering staff of the Howard Research and Development Corporation, developers of Columbia, with respect to land uses, aesthetic considerations, and acceptability. The responsibility for the exercise of judgement in qualitative matters rests, however, solely with the staff of Hittman Associates, who performed the analysis and made the final decisions.

It is interesting to note that the system actually selected from computer Run A, all-reuse-level option, was the thirty-first ranking system. Its total net benefit was \$20.79 per day lower than the highest-ranking system. Had the local reuse system been a factor in the original planning of this area, it is reasonable to assume that land planning would have provided for a system of significantly higher net benefit, thus improving the economics of the concept over those detailed in this report.

SYSTEMS ANALYSIS COMPUTER PROGRAM

The method employed to analyze and optimize the local reuse system has been described in detail in this and the previous chapter. The actual computations and organization of data referred to in the report were performed on an IBM 360-65 digital computer system, utilizing a computer code developed as part of this study.

SECTION XII

DEVELOPMENT OF CONCEPTUAL DESIGNS

The conceptual design and cost estimates of the Local Storage, Treatment, and Reuse system were prepared for three cases. These are:

1. Local Collection, Storage, and Treatment of Storm Water for Potable Reuse
2. Local Collection, Storage, and Treatment of Storm Water for Sub-Potable Reuse
3. Local Collection, Storage, and Treatment of Storm Water for Pollution Control

The system analysis and optimization studies discussed in the two previous chapters had provided a complete analysis of the various combinations of components and locations based on least overall cost. The system analysis program had not been programmed to evaluate actual site constraints and other design conditions. Accordingly, the development of conceptual designs and cost data involved (1) the selection of the system configuration and locations which could be constructed and operated in the Wilde Lake Watershed at the least overall cost, (2) the preparation of typical designs and drawings, (3) modification of costs based on actual site conditions, and (4) compilation of performance characteristics and cost data.

SELECTION OF PLANT LOCATIONS AND SYSTEM CONFIGURATIONS

A typical output of the system optimization program was previously shown as Figure 38, "Table of Net Benefits for Combinations of Watersheds," and was discussed in Section XI. Similar output sheets were produced for each of the three sectors of the Wilde Lake Watershed for which conceptual designs were desired. These tables provide a listing of the various combinations of plants and the net benefit (or cost) expressed in terms of dollars/day.

As a first step it was necessary to examine the topography, boundaries, and other site conditions to determine the maximum size reservoir that could be constructed at each possible location. This involved plotting the contours of the maximum water levels associated with various heights of dams, planimetry storage areas, estimating additional storage quantities that could be obtained by excavation, etc. In this examination, the existing land use plans and lot boundaries were considered invariant to assure a practical design based on actual site conditions.

The next step was to determine the available storage capacities of various combinations of reservoirs and to compare the available storage with the calculated volume for storms of one-year return interval. For each type of local storage system, the plant combination showing the maximum benefit or least cost was found from among the first feasible plant combination in the case of systems for potable reuse for the southwest section of the Wilde Lake watershed (Combination No.46, Figure 38).

Having selected the least cost and first feasible plant combination, the system configuration was also identified. The computerized system analysis program automatically calculates the capital and operating costs of alternative systems and selects the least cost configuration. Accordingly, it was only necessary to examine the printout of the program to determine the components that had been selected by the computer program for the various plants. In all cases, the least cost plant was based on the use of open storage reservoirs. The system analysis program also calculates and prints the amount of water supplied for reuse.

PREPARATION OF PLANT LOCATION AND TYPICAL DESIGN DRAWINGS

The conceptual design drawings showing the plant locations were prepared based on the selected least cost plant combinations. In the case of the "Local Collection, Storage, and Treatment of Storm Water for Sub-Potable Reuse," it was also necessary to lay out a distribution system. These drawings are contained in Sections XIII, XIV, and XV, with discussions of the conceptual designs of the three systems.

Since this system study also involved the preparation of conceptual design drawings of the facilities for a demonstration program, the locations for the demonstration facilities were used as the basis for the preparation of typical design drawings for storage reservoirs, pretreatment systems, and treatment plants.

MODIFICATION OF COSTS BASED ON ACTUAL SITE CONDITIONS

The typical design drawings of the facilities for the demonstration program were also used for another purpose. Detailed unit and material takeoffs were made and cost estimates based on the actual designs were prepared. These cost estimates were compared to the capital costs generated by the system analysis program. The capital cost inputs for the system analysis program had been based on parametric cost data as a function of reservoir capacity and treatment rates. Although all the major components were considered, it was not possible to consider all of the possible site conditions in this type of analysis. Based on the comparison of the computer-generated costs and the estimates based on the typical designs prepared for the demonstration program, the cost data used for the storage reservoirs and pretreatment units were found to be low. This was due to the following local conditions.

1. The storage reservoirs would be "Class 3 Impoundments" in accordance with recent regulations of the State of Maryland. "Class 3 Impoundments" are required to have spillways designed on the basis of a 100-year storm interval intensity. This regulation requires a dam of greater height and longer wing walls to retain the water. Approximately two feet were added to typical embankment height, and embankment lengths were generally doubled.
2. The reservoirs for the demonstration programs were designed to have a minimum pool depth of two feet to preclude weed growth. This, plus the "Class 3 Impoundment" requirement, increased the amount of excavation required.
3. The demonstration plant estimate includes fencing around the reservoirs.
4. The allowances for contingencies and engineering the demonstration plant are based on "first of a kind" facilities and the construction of a limited number of facilities.

The treatment plant designed for the demonstration program would probably not be typical of those used for the treatment of storm water in future installations, particularly where the quantity of water is relatively small. This plant is designed as a general purpose test plant in which it would be possible to evaluate various treatment processes. Once treatment processes are established for storm water, it is most likely that packaged treatment plants would be designed for this purpose. These would be automated plants designed to shutdown automatically upon failure and could be installed in either underground chambers or small surface structures.

For purposes of estimating the capital costs of constructing local storage, treatment, and reuse systems in the Wilde Lake Watershed, the estimated costs for the demonstration program storage reservoirs and treatment units were used in order to be conservative. This was done by plotting the estimates for the three demonstration reservoirs and pretreatment systems on the parametric cost curves used in the system analysis. New curves of costs vs. capacity were generated, and the appropriate costs were determined for each of the reservoirs required. Treatment plant costs were not modified. The treatment cost data used in the system analysis program are considered conservative, particularly since packaged treatment plants would be used.

COMPLILATION OF PERFORMANCE CHARACTERISTICS AND COST DATA

Each of the three local collection, storage, and treatment systems consists of a number of reservoirs, pretreatment, treatment, and distribution facilities. Accordingly, the conceptual design of the system

involved the compilation of the performance data for each of the individual installations and the evaluation of the overall effectiveness as a means of pollution control and water supply augmentation for the entire watershed. Similarly, the capital and operating costs of the individual installations were compiled to evaluate the overall cost of the system, the economic benefits of reuse, and the net costs of pollution control. These costs were then broken down to establish the costs per acre, per dwelling unit, and per thousand gallons of runoff. The performance and cost data for each of the three local collection, storage, and treatment systems is presented in the next three sections.

SECTION XIII

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORMWATER FOR POTABLE REUSE

The "Local Collection, Storage, and Treatment of Storm Water for Potable Reuse" system, hereafter referred to as the "Potable Reuse System," consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated sub-watersheds and treated water and overflows from the upstream storage reservoirs. Table 33 lists the locations of the 10 reservoirs and the sub-watersheds and upstream reservoirs from which flow is collected. The locations are shown on Figure 40.

TABLE 33. POTABLE REUSE SYSTEM
STORAGE RESERVOIR COLLECTION AREAS

Location	Sub-Watershed				Source of Flow	
					Upstream Reservoir	
A	1	2	3			-
B	11	12	13			A
C	4	5	14			D
D	6	7				E
E	8					-
F	9	15	16	17	18	20
G	19					B, C
H	22					-
I	21					J
J	10*					

*The only feasible location for a storage reservoir in sub-watershed 10 is Location J. At this location only 40 percent of the flow from the sub-watershed is intercepted.

A pretreatment unit is installed at each of the storage reservoirs. These pretreatment units treat the storm water to Class "C" quality for discharge to Wilde Lake or for further treatment and reuse. Treatment plants are located at nine of the storage reservoirs. The treatment plants take Class "C" water from the pretreatment units and treat the water to potable quality, Class "AA." Table 34 lists the type of storage, storage capacity, pretreatment capacity, quality of water produced, and the final treatment plant capacity. No treatment plant is provided at Location H because incremental costs of treatment exceeded possible benefits from reuse. Pretreatment units are sized for maximum flow independent of water diverted for treatment upstream.

Each of the nine treatment plants includes pumping facilities and connections to the public water systems. The treated storm water is supplied as a supplementary source of water and is distributed through the public water distribution systems to meet all types of demands, Classes "AA," "A," "B," and "C."

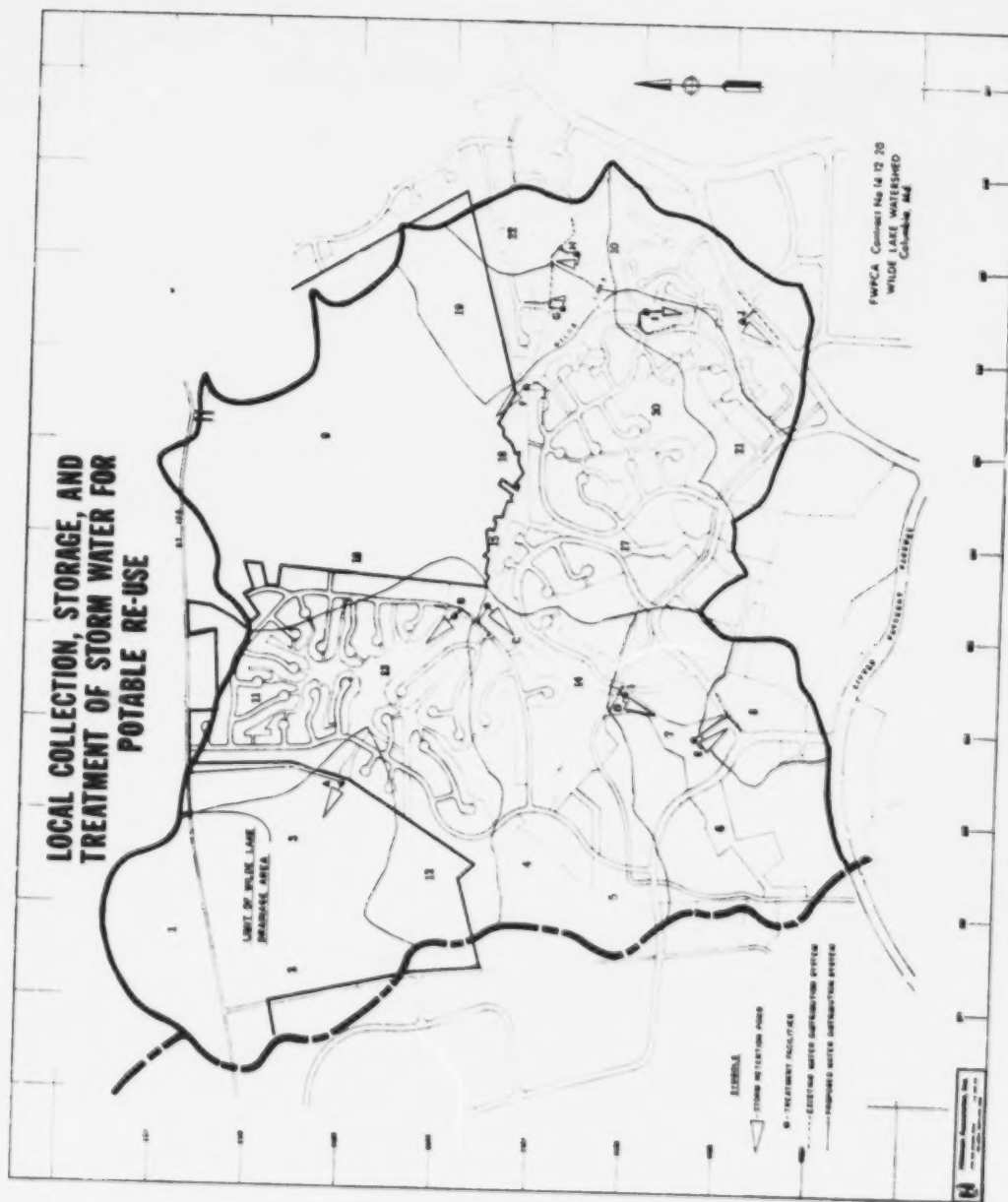


Figure 40. Potable Reuse System Location Plan

TABLE 34. LOCAL, COLLECTION, STORAGE, AND TREATMENT
OF STORM WATER FOR POTABLE REUSE - CONCEPTUAL DESIGN

Location	Storage	Capacity (Gal)	Pretreatment Capacity (gpd)	Water Quality Produced	Treatment Capacity (gpd)	Connected Demands
A	POND	1,577,300	67,900	AA	30,400	AA, A, B, C
B	POND	2,145,000	231,400	AA	81,800	AA, A, B, C
C	POND	1,902,100	386,100	AA	76,900	AA, A, B, C
D	POND	1,907,200	232,400	AA	86,800	AA, A, B, C
E	POND	625,900	58,900	AA	29,500	AA, A, B, C
F	POND	5,101,400	1,036,300	AA	209,400	AA, A, B, C
G	POND	923,500	85,700	AA	38,400	AA, A, B, C
H	POND	358,400	29,800	C		
I	POND	690,000	63,800	AA	31,900	AA, A, B, C
J	POND	503,200	55,100	AA	24,780*	AA, A, B, C

*40 percent of watershed intercepted by storage facility.

COMPONENT DESCRIPTIONS

Figure 41 is a flow diagram of a typical "Potable Reuse System" installation. Each of the installations (except at Location H) consist of the following:

Storage Reservoir

These reservoirs are open storage reservoirs, or ponds, formed by the construction of concrete dams.

Pretreatment Units

The pretreatment units are installed downstream and receive flow by gravity from the storm retention reservoirs. The pretreatment units consist of a float-operated valve, chlorination equipment chamber, and a tube settler. The pretreatment units have a sump for the collection of sediment and use float valves to control flow.

Treatment Plant

The treatment plant takes flow from the pretreatment unit. The plant consists of a mixed media filtration unit followed by an activated carbon column for carbon absorption. Provision is made for chemical feed prior to filtration and for chlorination prior to discharge to the treated water storage tank. The piping is arranged for backwashing the mixed media filter with a connection to the sanitary sewer.

Treated Water Storage

The treated water storage consists of steel tanks at grade. The treated water storage tank also serves as the filter backwash storage tank, the distribution pump clearwell, and the chlorine contact tank.

Pumping System

The pumping distribution system consists of a pump with associated control and a connection for supplying treated water to the public water system.

Sewer Connection

A connection is provided to public sewer systems to receive filter backwash. Appropriate measures are taken to prevent any possibility of backflow from the sewer.

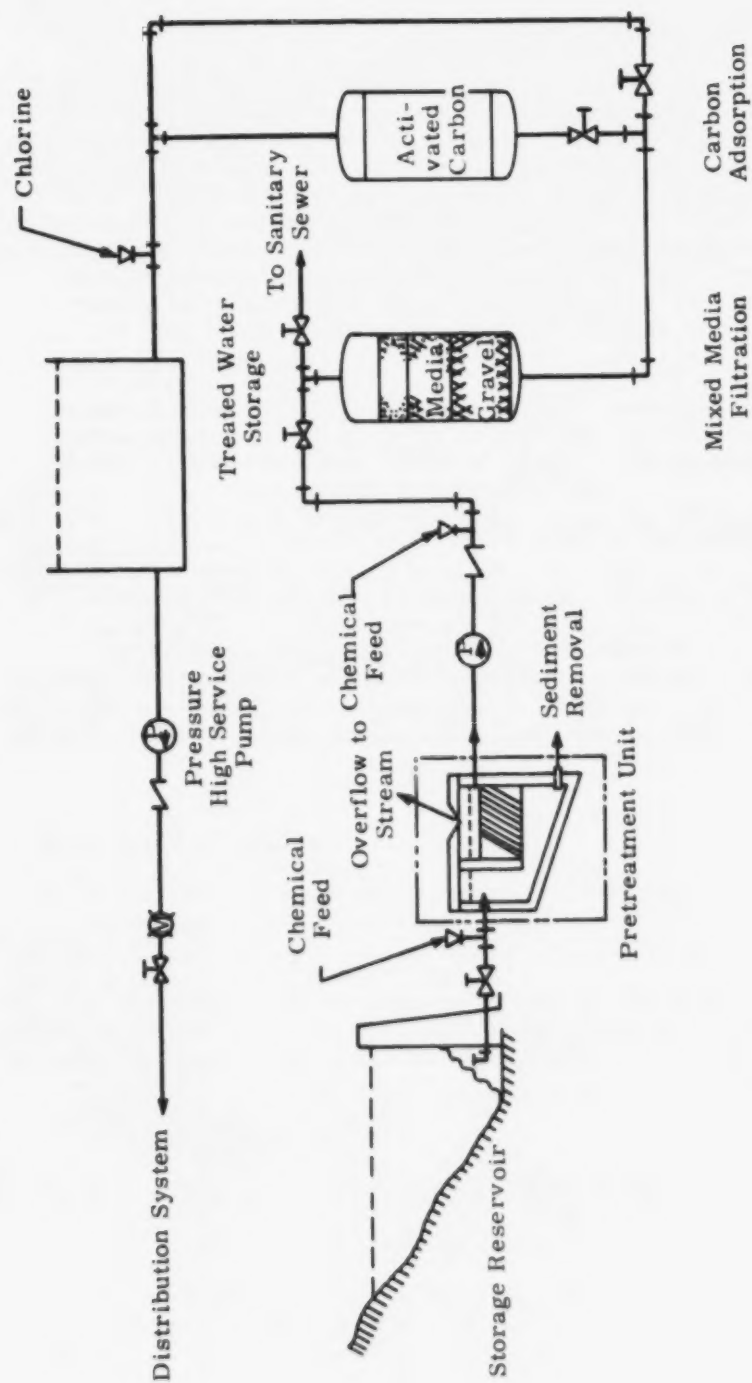


Figure 41. Potable Reuse System Flow Diagram

PERFORMANCE CHARACTERISTICS

The demands for potable water in the Wilde Lake Watershed will exceed the amount of storm water runoff available even with the higher runoff quantities that will result when the area is further developed. In addition, since the treated storm water would be supplied to and distributed by the public water system, the demands of any part of the watershed can be supplied. This is contrasted to systems using a separate distribution system in which the supply and demand of the individual sub-watershed must be considered. Table 35 contains a listing of the annual potable water requirements, the total amount of storm water runoff, and the amount of treated water supplied to the service areas from the treatment plants.

Since the demands of the watershed exceed the available supply, the "Potable Reuse System" was designed to produce the maximum treated water in order to maximize the reuse benefit. An upper limit of 90 percent of the available storm runoff was used in the design of the individual reservoirs and treatment plants, in order to assure that a portion of the storm runoff would still go to Wilde Lake, in addition to the base flow. As finally designed, the "Potable Reuse System" will provide 202,942,000 gallons per year of calculated annual runoff of 246,967,000 gallons. This represents 82 percent of the available supply. About 29,000,000 gallons of treated water would be released to Wilde Lake and an additional 15,000,000 gallons of storm water runoff would go directly to the lake since the only possible location for a reservoir in sub-watershed 10 is at Location J which can only intercept some 40 percent of the runoff from the sub-watershed.

The pretreatment units are designed to treat storm water at twice the mean annual daily runoff rate, $2 \times \bar{q}$. In order to maximize the treated water supply, the pretreatment units would be operated at 1.1 to $1.4 \times \bar{q}$, plus the base flow. At this rate, there is an increased probability of not having the reservoir emptied sufficiently to receive the runoff from a subsequent storm. In terms of the design criteria of storms of one-year recurrence intervals, the $2 \times \bar{q}$ discharge rate would retain 98.5 percent of the runoff, whereas a $1.13 \times \bar{q}$ discharge rate would retain 97.3 percent of the flow. Expressed in terms of all storms including those greater than one-year recurrence interval, the following relationships were calculated.

<u>Condition</u>	<u>Percent of Total Runoff Retained</u>
One-year storm design - no residual assumed	92.6
One-year storm design - $2 \times \bar{q}$ discharge rate	91.2
One-year storm design - $1.13 \bar{q}$ discharge rate	90.0

Accordingly, with the "Potable Reuse System" not more than 10 percent of the runoff would normally overflow the reservoir and not receive the full Class "C" treatment. However, even the overflows will be retained in the storage reservoirs for a time period adequate to allow settlement

TABLE 35. POTABLE WATER REQUIREMENTS, TOTAL STORM WATER RUNOFF, AND AMOUNT SUPPLIED
(Amounts in Gallons/Year)

Treatment Plant Location	Sub-watersheds	Potable Water Requirement		Total Storm Water Runoff		Amount Supplied
		By Sub-watershed	Pretreatment Plant	By Watersheds	Per Treatment Plant	
A	1	4,519,000		3,627,000		
	2	4,551,000		4,694,000		
	3	1,858,000	10,928,000	4,073,000	12,394,000	11,104,500
B	11	15,283,000		12,378,000		
	12	7,234,000		7,332,000		
	13	11,852,550	34,369,000	10,134,000	29,844,000	26,754,500
C	4	9,278,000		4,791,000		
	5	10,289,000		7,733,000		
	14	24,229,000	43,796,000	15,528,000	28,052,000	25,155,800
D	6	67,435,000		17,653,000		
	7	35,679,000	103,114,000	14,007,000	31,660,000	28,185,300
	8	16,542,000	16,542,000	10,741,000	10,741,000	9,621,400
F	9	22,907,000		17,818,000		
	15	3,927,000		3,700,000		
	16	15,830,000		10,260,000		
G	17	24,794,000		20,388,000		
	18	6,216,000		3,905,000		
	20	20,122,000	93,796,000	20,351,000	76,422,000	68,612,700
H	19	8,399,000	8,399,000	15,637,000	15,637,000	14,009,900
	22	7,296,000	7,296,000	5,440,000	5,440,000	
I	21	22,601,000	22,601,000	11,653,000	11,653,000	10,453,600
	10	45,045,000	45,045,000	25,124,000	10,049,000*	9,044,500
Totals		385,886,000	385,886,000	246,967,000	231,892,000	202,942,200

*Intercepts 40 percent of the sub-watershed.

of the heavier particulate matter. Added to the runoff that cannot be practically intercepted, this would represent 22,000,000 to 40,000,000 gallons per year compared to the 247,000,000 gallons or greater amount of runoff that would otherwise be discharged directly to the lake. Neither estimate includes the base flow from the watershed which is not affected by individual storms.

Most of the water reused by the "Potable Reuse System" would eventually return to the river basin as effluent from the wastewater treatment plant. Since this is downstream from Wilde Lake, the remaining flow of the lake must also be considered. Water must be supplied to Wilde Lake to provide for the evaporative losses, seepage from the lake, and the release requirements placed on the Wilde Lake dam by the State of Maryland. The evaporative losses of the lake were calculated to be about 22,000,000 gallons per year. However, the rainfall directly into the lake represents an equal or slightly greater amount of water. The state requirement on release is a general requirement that precludes diversion of the water from the lake itself.

The 29,000,000 gallons of treated water and the 22,000,000 to 40,000,000 gallons of untreated water that would be discharged to the lake did not include the base flow from the stream. As discussed in Section II, a temporary gaging station was installed and operated on a 130-acre sub-watershed in Wilde Lake as part of this study. The measured base flow of 0.05 cfs, extrapolated to the entire 1140-acre watershed, would indicate an annual base flow to the lake in excess of 104,000,000 gallons per year. The total flow of 150 to 170 million gallons of water per year to Wilde Lake would correspond quite closely to the natural condition of the area with a runoff calculated at 43,000,000 gallons per year plus the base flow.

DESIGN AND CONSTRUCTION COSTS

The "Potable Reuse System" will consist of 10 storage reservoirs and pretreatment units and nine storm water treatment plants, pumping systems, and connections to the public water and sewer system. The major design and construction items are summarized in Table 36.

As discussed previously, land costs were not included on the basis that all of the storage reservoirs and treatment plants could be located within the floodplain which is dedicated to open space in the case of Columbia.

In order to provide conservative estimates of the design and construction costs for the "Potable Reuse System" for comparison with other systems, the cost estimates in Table 36 are based upon the assumptions and criteria used in the system analysis. As noted previously in Section XII, the estimated costs of the storage reservoirs and pretreatment units have been scaled up to take full account of local conditions. It is considered that these costs could be reduced substantially by further design work. Likewise, estimates for the treatment plants are based on small size conventional treatment plants. It is envisioned that packaged treatment plants

would be developed for future "Potable Reuse Systems," and these costs would also be substantially reduced.

OPERATION AND MAINTENANCE COSTS

The operation and maintenance costs for the "Potable Reuse System" were estimated using the assumptions and criteria used in the system analysis as discussed in Appendices C and D. Since a number of nearly identical installations are involved which would be located in a relatively small area, the parametric cost estimates were modified based on the actual designs that had been developed. For example, checks and operations can be performed by personnel moving from one location to another. Likewise, maintenance items such as the cleaning of sediment from reservoirs can be accomplished by a crew handling a number of facilities at one time. Certain of the costs (i.e., chemicals, pumping power, etc.) can be directly related to the quantities of water processed. As discussed in Appendix D, the costs for operation and maintenance, as based on this work being performed by extension and expansion of an existing organization and a supporting organizational and overhead structure, has not been included.

Table 37 contains a summary of the operation and maintenance for the "Potable Reuse System."

TABLE 36. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM
WATER FOR POTABLE REUSE - DESIGN AND CONSTRUCTION COSTS
(Amounts in Dollars)

Construction	A	B	C	D	Location				I	J	Total
					E	F	G	H			
Storage Reservoirs	61,000	71,500	67,500	68,000	40,500	109,000	47,000	32,500	42,000	33,500	
Pretreatment Units	8,200	9,600	10,200	9,600	8,000	11,400	8,600	7,300	8,200	7,600	
Treatment Plant	35,600	41,800	41,100	42,400	35,400	78,400	36,600		35,800	35,000	
Pumping and Distribution	12,400	12,320	12,600	12,800	10,200	19,400	10,800		10,600	12,100	
Subtotal	117,200	135,200	131,400	132,800	94,100	218,200	103,000	39,800	96,600	87,600	1,155,900
Design, Inspection and Field Engineering	17,500	20,300	19,700	20,000	14,100	32,700	15,600	3,000	14,600	13,200	173,700
Contingencies and Escalation	11,700	13,500	13,100	13,300	9,400	21,800	10,300	4,000	9,700	8,800	115,600
Total	146,400	169,000	164,200	166,100	117,600	272,700	128,900	49,800	120,900	109,600	1,445,200

TABLE 37. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM
WATER FOR POTABLE REUSE - ANNUAL OPERATING AND MAINTENANCE COSTS

	Labor (man hours)	Labor (\$)	Material (\$)	Equip. Rental* (\$)	Miscellaneous (\$)	Totals
Storage Reservoirs						4,090
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690	250	2,400		
(3) Sediment Removal						
Pretreatment Units						10,110
(1) Routine Checks	260	750		3,250		
(2) Sediment Removal			4,500			
(3) Chemicals						
(4) Preventative Mainte- nance and Repair	610	1,760	880			
Treatment Plants						35,030
(1) Routine Inspections	2,920	8,400	400			
(2) Water Tests	940	3,060				
(3) Routine Operation	1,880	5,400	2,500			
(4) Chemicals						
(5) Preventative Mainte- nance	2,350	7,640	3,820		3,000 810	
(6) Electrical Energy						
(7) Sewer Change						
Pumping Facility						6,390
(1) Routine Checks	240	715				
(2) Preventative Mainte- nance and Repairs	700	2,260	1,130		3,000	
(3) Electrical Energy						
(4) Distribution		120				
Miscellaneous			14,090			14,090
Totals	10,436	31,545	26,840	5,650	6,810	70,845

*Include Operators

SECTION XIV

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR SUB-POTABLE REUSE

The "Local Collection, Storage, and Treatment of Storm Water for Sub-Potable Reuse" system, hereafter referred to as the "Sub-Potable Reuse System," consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated sub-watersheds and treated water from the upstream storage reservoirs. Table 38 lists the locations of the 10 storage reservoirs and the sub-watersheds and upstream reservoirs from which flow is collected. The locations are shown on Figure 42.

**TABLE 38. SUB-POTABLE REUSE SYSTEM
STORAGE RESERVOIR COLLECTION AREAS**

<u>Source of Flows</u>							
<u>Location</u>	<u>Sub-watersheds</u>						<u>Upstream Reservoirs</u>
A	1	2	3				-
B	11	12	13				A
C	4	5	14				D
D	6	8					E
E	7						-
F	9	15	16	17	18	20	B, C
G	19						
H	22						
I	21						
J	10 *						

*Reservoir J only intercepts 40 percent of sub-watershed 10.

The "Sub-Potable Reuse System," like the "Potable Reuse System," has pretreatment units which treat the storm water to Class "C" quality for discharge to Wilde Lake or for further treatment. In the case of the "Sub-Potable Reuse System" the water from the pretreatment units is either distributed as Class "B" or Class "C" water. In the case of Class "B" reuse, a treatment plant is installed with a pumping station which takes the water from Class "C" to Class "B" quality. In the case of



Figure 42. Sub-Potable Reuse System - Location Plan

Class "C" reuse, the water is taken from the pretreatment units through a pumping station. As shown on Figure 42, a distribution system is provided for the sub-potable water in each sub-watershed. Connections are provided from the pumping stations to the distribution system in the sub-watershed or combination of sub-watersheds from which runoff is collected.

Table 39 lists the type of storage, storage capacity, pretreatment capacity, water quality produced, final treatment quality, and the connected demands for each of the reservoirs and treatment plants used with the "Sub-Potable Reuse System."

Comparing Table 33 and Table 38 it is noted that the "Potable Reuse System" and the "Sub-Potable Reuse System" differ with respect to the collection areas of reservoirs D and E. As shown on Figure 38, the optimal feasible case for the "Potable Reuse System" is the Combination Number 46 (Cases 14, 11, and 5), whereas the optimal sub-potable combination is number 48 (Case 14, 10, and 4) for Class "C" water.

COMPONENT DESCRIPTIONS

The components comprising the "Sub-Potable Reuse System" installations are similar to those of the "Potable Reuse System" described in Section XIII. The two types of systems differ only with respect to the treatment plants and the sub-potable distribution system.

The "Sub-Potable Reuse System" has two types of treatment facilities. Figure 43 shows a flow diagram for a Class "B" treatment plant of the type used at Locations A, B, C, and F. For this type plant, water from the pretreatment unit goes through a microstrainer and is pumped to a treated water storage tank which also serves as a chlorine contact tank. A high pressure service pump supplies the sub-potable distribution system. A connection from the public water supply system provides water when the local treatment plant is not operating.

Figure 44 is a flow diagram for a Class "C" treatment plant of the type used at Locations D, E, G, H, I, and J. With this type plant, flow from the pretreatment unit goes directly to a treated water storage tank which is also a chlorine contact tank and is then pumped to the sub-potable distribution system. A connection from the public water supply is also provided to provide water when the plant is not operating.

The sub-potable distribution system consists of underground pipe lines as shown on Figure 42 and the separate piping systems within the houses, town houses, apartments, schools, and commercial buildings. The design is based on the construction of the sub-potable distribution system following installation of other utilities. In actual practice, such a system might be placed at the same time as the public water supply and sewer system. This would appreciably reduce the cost of the sub-potable system.

TABLE 39. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR SUB-POTABLE REUSE - CONCEPTUAL DESIGN

Location	Type of Storage	Storage Capacity (Gal)	Pretreatment Capacity (gpd)	Water Quality Produced	Final Treatment Capacity (gpd)	Connected Demands
A	POND	1,577,300	67,900	B	23,500	B, C
B	POND	2,145,000	231,400	B	54,000	B, C
C	POND	1,902,100	386,100	B	65,400	B, C
D	POND	803,000	232,400	C	33,600	C
E	POND	1,729,900	155,600	C	77,800	C
F	POND	5,101,400	1,036,300	B	160,200	B, C
G	POND	923,500	85,700	C	4,300	C
H	POND	358,400	29,800	C	5,400	C
I	POND	690,000	63,800	C	20,100	C
J	POND	503,000*	55,100	C	20,700	C

*40 percent of watershed intercepted by storage facility.

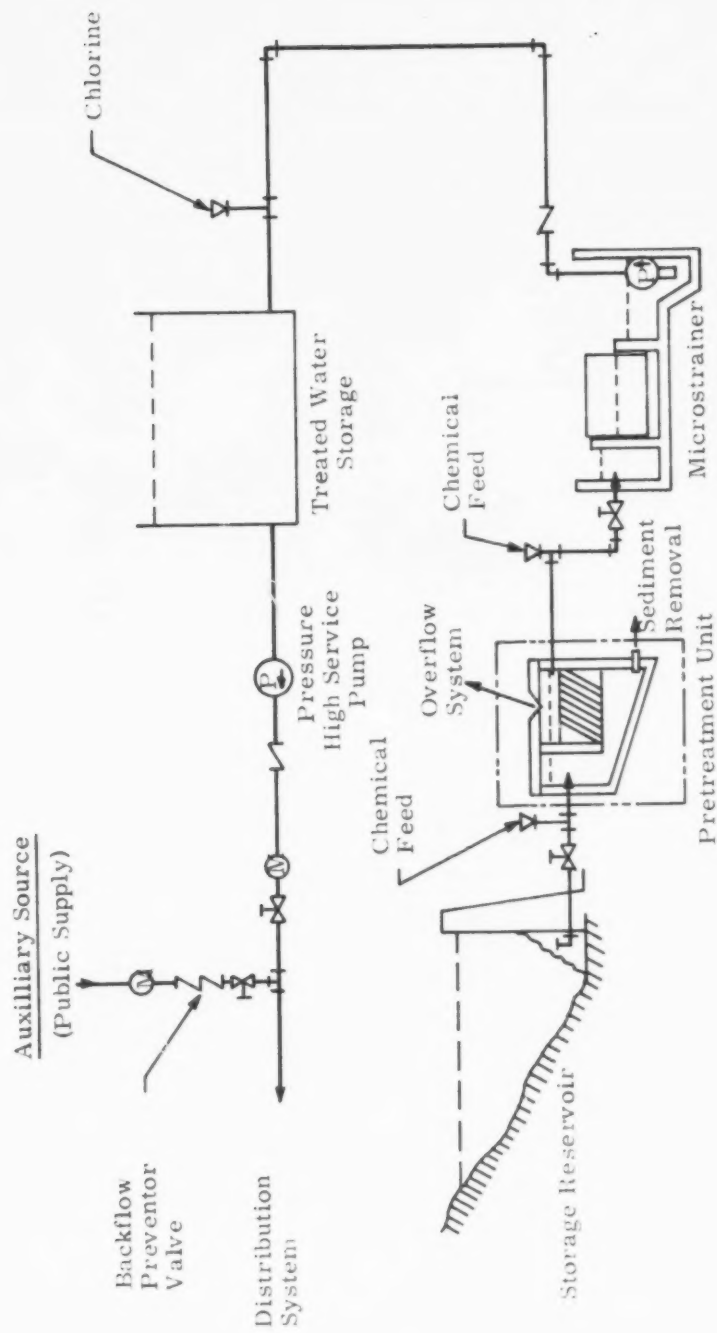


Figure 43. Sub-Potable Reuse System Flow Diagram - Class "B" Treatment

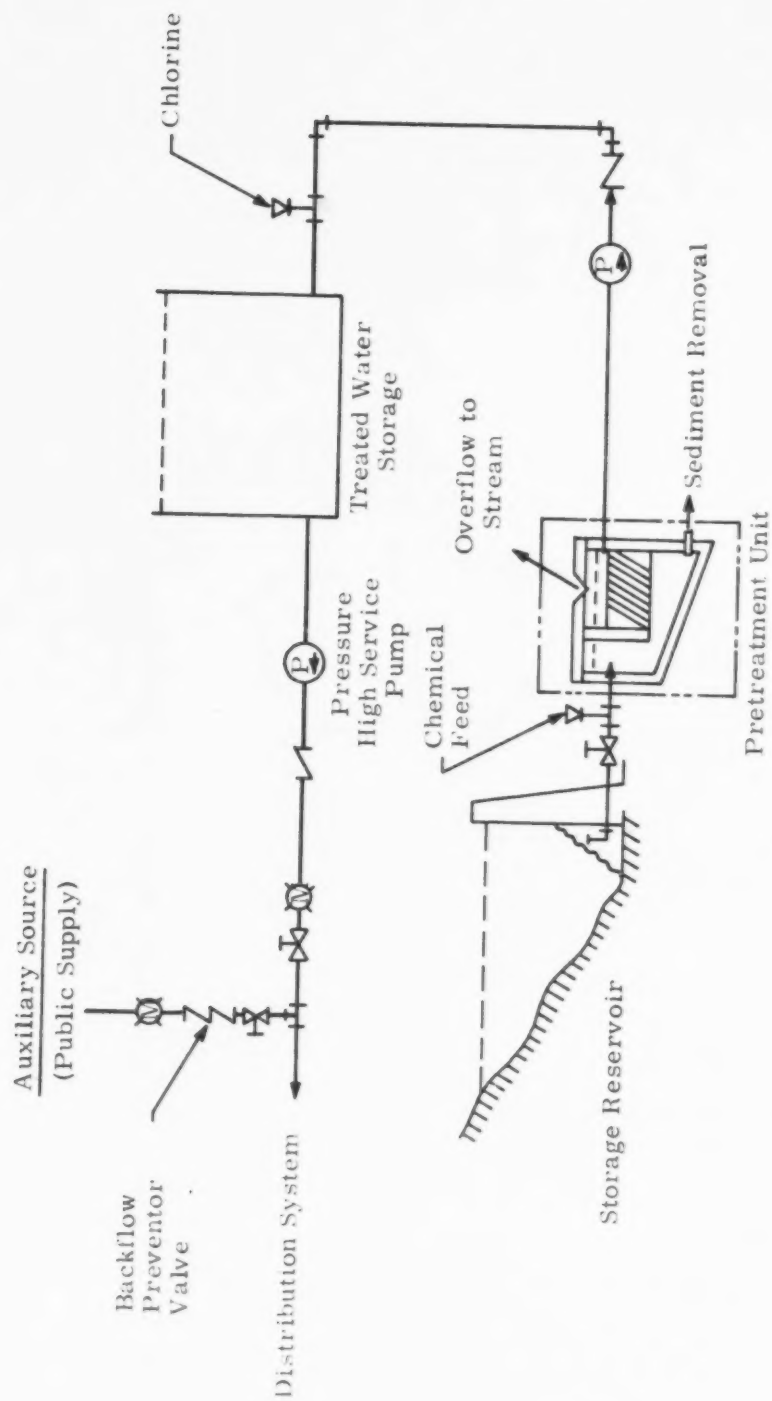


Figure 44. Sub-Potable Reuse System Flow Diagram -
Class "C" Treatment

PERFORMANCE CHARACTERISTICS

In the case of the "Sub-Potable Reuse System," the water is distributed for use to the sub-watersheds from which it is collected. Unlike the "Potable Reuse System," the "Sub-Potable Reuse System" must consider the available supply of water and the demands for sub-potable water on a sub-watershed by sub-watershed basis. Table 40 contains a listing of the sub-potable water requirements by class of water to be distributed for each sub-watershed and treatment plant, the total storm water runoff available from each sub-watershed at each treatment plant, and the amount of water supplied.

The amount of water supplied for reuse in the Wilde Lake watershed would be 167,849,000 gallons per year. This represents 46.5 percent of the total water requirements of the area.

The pretreatment units would be operated in the same manner as previously described for the "Potable Reuse System." The probability of spills and quantities of water overflowing the reservoirs or going directly to Wilde Lake without the full Class "C" treatment would be essentially the same for the "Sub-Potable Reuse System."

With respect to maintaining adequate flow to Wilde Lake, the "Sub-Potable Reuse System" would normally discharge about 70,000,000 gallons of treated water to the lake. This would be in addition to the base flow, possible overflows, and the 15,000,000 gallons from sub-watershed 10 that cannot be intercepted in a practical manner. Accordingly, more than adequate flow will be maintained to the lake.

The 167,849,000 gallons of water supplied to the area represents an annual benefit of \$75,532 at the current distribution price of 45 cents per thousand gallons.

DESIGN AND CONSTRUCTION COSTS

The major construction items for the "Sub-Potable Reuse System" will be the same as those for the "Potable Reuse System" in Section XIII with the following exceptions:

1. As discussed above, the treatment plants will have fewer components
2. The sub-potable distribution system, composed of:
 - a. Underground piping
 - b. Dual systems within buildings

Table 41 contains a summary of the estimated design and construction costs for the "Sub-Potable Reuse System." As shown, the cost of the

TABLE 40. SUB-POTABLE WATER REQUIREMENTS, TOTAL
STORM WATER RUNOFF, AND AMOUNT SUPPLIED
(Amounts in Gallons/Year)

Treatment Plant Location	Sub- Watersheds	Class	Sub-Potable Water Requirement		Total Storm Water Runoff	
			by Sub-watersheds Quantities	Per Treatment Plant	by Watershed	per Treatment Plant
A	1	B, C	3,369,000		3,627,000	
	2		3,401,000		4,694,000	
	3		1,420,000	8,190,000	4,073,000	12,394,000
B	11	B, C	8,658,000		12,378,000	
	12		4,278,000		7,332,000	
	13		6,924,000	19,860,000	10,134,000	29,844,000
C	4	B, C	5,212,000		4,791,000	
	5		4,923,000		7,733,000	
	14		13,724,000	23,859,000	15,528,000	28,052,000
E	6	C	23,327,000		17,653,000	
	8		5,497,000	28,824,000	10,741,000	28,394,000
D	7	C	12,238,000	12,238,000	14,007,000	14,007,000
F	9	B, C	17,166,000		17,818,000	
	15		2,230,000		3,700,000	
	16		10,957,000		10,260,000	
G	17	C	12,038,000		20,388,000	
	18		3,697,000		3,905,000	
	20		12,363,000	58,451,000	20,351,000	76,422,000
H	19	C	1,540,000	1,540,000	15,637,000	15,637,000
I	22	C	1,964,000	1,964,000	5,440,000	5,440,000
J	21	C	7,315,000	7,315,000	11,653,000	11,653,000
Totals	10	C	12,687,000	12,687,000	25,124,000	10,049,000*
			174,928,000	174,928,000	246,967,000	231,892,000
						167,849,000

*Based on intercepting 40 percent of sub-watershed

TABLE 41. LOCAL COLLECTION, STORAGE, AND TREATMENT
OF STORM WATER FOR SUB-POTABLE REUSE
DESIGN AND CONSTRUCTION COSTS
(Amounts in Dollars)

Construction	Location									
	A	B	C	D	E	F	G	H	I	J
Storage Reservoirs	61,000	71,500	67,500	45,000	65,000	109,000	47,000	32,500	42,000	33,500
Pre-Treatment	8,200	9,600	10,200	9,600	9,200	11,400	8,600	7,300	8,200	7,600
Treatment Plants	15,000	15,900	16,200			18,700				
Pumping System and Connection	12,700	11,200	13,300	5,700	5,800	27,700	5,500	5,400	5,900	7,800
Subtotal	96,900	106,200	107,200	60,300	80,000	166,800	61,100	45,200	56,100	48,900
Distribution	37,400	134,800	143,400	230,000	100,400	380,000	43,600	25,200	57,900	97,100
Subtotal	134,300	241,000	250,600	290,300	180,400	546,800	104,700	71,400	114,000	146,000
Design, Inspection and Field Engineering	21,100	36,200	37,700	43,500	27,600	81,100	15,800	10,800	17,100	21,900
Contingencies and Escalation	13,400	24,100	25,100	29,000	18,400	54,700	10,500	7,200	11,400	14,600
Total	168,800	301,300	312,800	362,800	225,400	682,600	136,000	87,900	142,500	182,500
										2,598,600

sub-potable distribution system represents a major portion of the cost of the overall system. The design of the sub-potable system is based on a system of comparable design to public water systems and assumes that it is installed as a separate entity. As previously noted, in the case of new developments, this system would probably be installed concurrently with the public water and sewer system. In other cases where there is a large irrigation or industrial demand for sub-potable water, a simpler or less expensive distribution system could be used. Table 42 illustrates the limits and effect of the cost of the distribution system.

OPERATION AND MAINTENANCE COSTS

Table 43 contains a summary of the operation and maintenance costs for the "Sub-Potable Reuse System."

The operation and maintenance costs for the "Sub-Potable System" were estimated on the same basis as the "Potable Reuse System" with treatment plant operating costs being lower and an allowance made for the maintenance of the distribution system.

TABLE 42. EFFECT OF DISTRIBUTION SYSTEM ON CAPITAL COSTS

System without Distribution Costs

Construction Costs without Distribution	\$ 828,700.00
Design, Contingencies, and Escalation (25%)	207,200.00
Subtotal without Distribution	1,035,900.00
Equivalent Cost per Year	54,220.00
Equivalent Cost per Day	148.54

System with Distribution Costs

Construction Cost with Distribution	2,074,200.00
Design, Contingencies, and Escalation	524,400.00
Total with Distribution	2,598,600.00
Equivalent Cost per Year	136,013.00
Equivalent Cost per Day	372.64
Value of Sub-Potable Water per Year	75,532.00
Value of Sub-Potable Water per Day	206.94

TABLE 43. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER
FOR SUB-POTABLE REUSE - ANNUAL OPERATION AND MAINTENANCE COSTS

	Labor (man hour)	Labor (\$)	Materials (\$)	Equipment Rental (\$)	Utilities (\$)	Totals 4,090
Storage Reservoirs						
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690	250			
(3) Sediment Removal				2,400		
Pretreatment Units						10,050
(1) Routine Checks	260	750				
(2) Sediment Removal				3,250		
(3) Chemicals			4,500			
(4) Preventative Mainte- nance and Repair	520	1,730	850			
Treatment Plants						18,520
(1) Routine Inspection	1,460	4,200				
(2) Water Testing	1,040	3,060				
(3) Routine Operations	1,454	4,200				
(4) Chemicals			3,000			
(5) Preventative Mainte- nance and Repair	406	1,320	660		1,680	
(6) Electrical Energy					500	
(7) Sewer Charge						
Pumping Facilities						5,340
(1) Routine Checks	312	900				
(2) Preventative Mainte- nance and Repair	394	1,280	640			
(3) Electrical Energy					2,400	
(4) Connections	36	120				
Distribution System						7,500
(1) Maintenance and Repair	1,054	3,750	3,750			
Miscellaneous			8,670			8,670
Total	69,361	22,750	22,320	5,650	4,580	55,300

SECTION XV

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR POLLUTION CONTROL

The "Local Collection, Storage, and Treatment of Storm Water for Pollution Control" system, hereafter referred to as the "Local Pollution Control System," consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated watershed and treated water and overflows from upstream reservoirs. Each storage reservoir is connected to a pretreatment unit and the water collected by the reservoir is treated to Class "C" quality and discharged into the stream beds and ultimately to Wilde Lake. Figure 45 shows the locations of the reservoirs and the pretreatment units. Table 44 lists the annual runoff from each sub-watershed and the amount annually collected by each reservoir. Table 44 has been arranged to illustrate the interaction of the individual installations and the amount of flow coming from upstream reservoirs, the total flow treated annually and average per day, and the sizing of the pretreatment units and the reservoirs.

Unlike the "Potable Reuse System" and the "Sub-Potable Reuse System" described previously, the "Local Pollution Control System" makes no provision for the treatment of water beyond Class "C" quality or for distributing the water. The conceptual design for the "Local Pollution Control System" is intended to provide a basis for comparison with other pollution control systems and a reference for evaluating the benefits of reuse. The "Local Pollution Control System" has been optimized and conceptually designed as a system that could be applied for pollution control independent of reuse consideration.

COMPONENT DESCRIPTIONS

The "Local Pollution Control System" installations consist of two components, a storage reservoir and a pretreatment unit. These components are essentially identical in design to the storage reservoirs and pretreatment units previously described for the "Potable Reuse System" and the "Sub-Potable Reuse System." The criteria used in sizing the reservoirs and pretreatment units for the "Local Pollution Control System" are slightly different since none of the water is diverted for reuse. With the "Local Pollution Control System," flow leaving the pretreatment unit after receiving treatment to Class "C" quality is discharged directly to the stream beds.

PERFORMANCE CHARACTERISTICS

Since the sole purpose of the system is pollution control, the pretreatment units would be operated at a higher rate than the two reuse systems previously discussed. In the case of reuse systems, pretreatment

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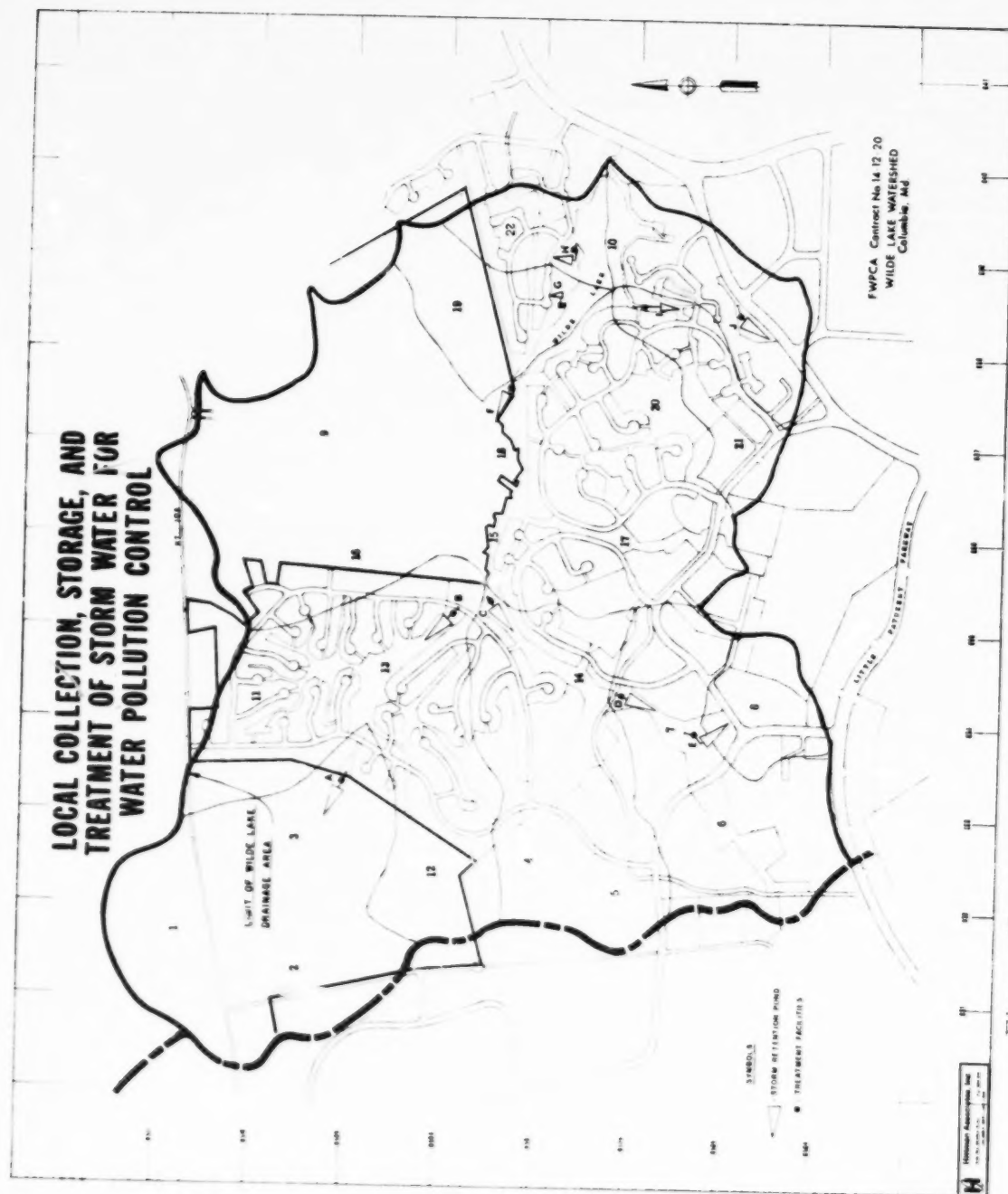


Figure 45. Local Pollution Control System - Location Plan

TABLE 44. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER
FOR POLLUTION CONTROL RUNOFF BY SUB-WATERSHED AND PRETREATMENT UNIT
WITH PRETREATMENT AND RESERVOIR CAPACITIES

Reservoir/ Pretreatment Location	Sub-Watershed Runoff No.	Sub-Watershed Runoff (gpy)	Total Runoff Per Reservoir (gpy)	Flow from Upstream Reservoir No.	Total Flow Treated (gpy)	Pretreatment Capacity (gpd)	Reservoir Size (gals)
A	1	3,627,000	12,394,000		12,394,000	68,000	1,577,300
	2	4,694,000					
	3	4,073,000					
B	11	12,378,000	29,844,000	A	12,394,000	232,000	2,145,000
	12	7,332,000					
	13	10,134,000					
E	8	10,741,000	10,741,000		10,741,000	59,000	1,729,000
D	6	17,653,000	31,660,000	E	42,401,000	232,400	803,000
	7	14,007,000					
C	4	4,791,000	28,052,000	D	42,401,000	386,000	1,902,000
	5	7,733,000					
	14	15,528,000					
F	9	17,818,000	76,422,000	B	42,238,000	1,036,000	5,101,000
	15	3,700,000					
	16	10,260,000					
	17	20,388,000					
	18	3,905,000					
	20	20,351,000					
G	19	15,637,000	15,637,000	C	42,238,000	86,000	923,500
H	22	5,440,000	5,440,000		15,637,000	29,800	358,400
I	21	11,653,000	11,653,000		5,440,000	64,000	690,000
J	10	25,124,000	10,049,000*		11,653,000	53,200	375,000*
		246,967,000			10,049,000*		

*Based on intercepting 40 percent of the watershed

flow rates would be regulated to maximize the water available for reuse. The "Local Pollution Control System" would be operated at a rate of twice the mean annual daily flow rate to increase the probability that the reservoirs will be able to collect the majority, if not all, of subsequent storms. As discussed in Section XIII, with the pretreatment unit operating at twice the mean annual runoff plus base flow, 98.5 percent of all one-year storms will be stored. At this rate 91.2 percent of all storms, including those with return intervals greater than one year, will be retained.

With the "Local Pollution Control System" only that water from sub-watershed number 10 will enter Wilde Lake untreated. This amounts to 15,075,000 per year and of the remaining 231,892,000 gallons of annual runoff, only 3,478,000 gallons will not receive the full Class "C" treatment for any normal year and only 20,407,000 gallons for storms of greater return intervals. All water except that from sub-watershed 10 will be retained in the storage reservoirs for a sufficient interval to permit the heavy particles to settle out. Expressed differently, if the water not receiving full treatment were to have a suspended solids concentration of 200 ppm, the resultant overall suspended solids in the lake would be 43 ppm when diluted with the treated effluent having 30 ppm suspended solids, assuming no sedimentation in Wilde Lake itself.

DESIGN AND CONSTRUCTION COSTS

The construction items for the "Local Pollution Control System" would be the same as the two reuse systems previously discussed with respect to the storage reservoirs and pretreatment units. Table 45 contains a summary of the estimated design and construction costs for the "Local Pollution Control System."

OPERATION AND MAINTENANCE COSTS

The operation and maintenance for the "Local Pollution Control System" would involve the same items of work as the storage reservoirs and pretreatment units for the two reuse systems previously discussed. Table 46 contains a summary of the estimated costs for the "Local Pollution Control System."

TABLE 45. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER
FOR WATER POLLUTION CONTROL - DESIGN AND CONSTRUCTION COSTS

	<u>A</u>	<u>B</u>	<u>C</u>	<u>D</u>	<u>Location</u>			<u>I</u>	<u>J</u>	<u>Total</u>
					<u>E</u>	<u>F</u>	<u>G</u>			
Construction										
Storage Reservoirs	61,000	71,500	67,500	45,000	65,000	109,000	47,000	32,500	42,000	33,500
Pretreatment Units	8,200	9,600	10,200	9,600	9,200	11,400	8,600	7,300	8,200	7,600
Subtotal	69,200	81,100	77,700	54,600	74,200	120,400	55,600	39,800	50,200	41,100
										663,900
Design, Inspection and Field Engineering	10,400	12,100	11,700	8,200	11,100	18,000	8,400	6,000	7,500	6,100
										99,500
Contingencies and Escalation	6,900	8,100	7,800	5,500	7,400	12,000	5,600	4,000	5,000	4,100
										66,400
Total	86,500	101,300	97,200	68,300	92,700	150,400	69,600	49,800	62,700	57,300
										829,800

TABLE 46. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR
WATER POLLUTION CONTROL - ANNUAL OPERATION AND MAINTENANCE COSTS

	Labor (man hrs.)	Labor (\$)	Material (\$)	Equipment Rental (\$)	Utilities (\$)	Totals
Storage Reservoirs						
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690				
(3) Sediment Removal				2,400		
Pretreatment Units						
(1) Routine Checks	260	750				
(2) Sediment Removal				3,250		
(3) Chemicals			4,500			
(4) Preventative Maintenance and Repair	520	1,730	1,730			
(5) Electrical Energy					200	
Miscellaneous						
			3,600			
Subtotals	1,280	4,320	9,830	5,650	200	\$20,000

SECTION XVI

CONVENTIONAL STORM WATER TREATMENT SYSTEM

In order to evaluate the systems for "Local Storage, Treatment, and Reuse of Storm Water" it was necessary to develop a base for comparison. As a parallel effort, a conceptual design was prepared for a system to control the pollution of Wilde Lake using conventional design methods. This work was performed by Whitman, Requardt and Associates of Baltimore, Maryland. A summary description of the Conventional Storm Water Treatment System, hereafter referred to as the "Conventional Treatment System," is contained in this chapter.

DESIGN REQUIREMENTS

The design of the Conventional Treatment System was based on the same area, runoff quantities, flow hydrographs, and same quality of water requirements as the Local Storage, Treatment, and Reuse System. Table 47 contains a summary of the design requirements.

TABLE 47. DESIGN REQUIREMENTS FOR
CONVENTIONAL TREATMENT PLANT

Area Considered:	Wilde Lake Watershed
Treated Water Quality:	State of Maryland Water Quality Standards for "Water Contact Recreation" modified as discussed in Section V under Class "C" water quality
Water Quantity:	Runoff quantities as calculated using the synthesized hydrograph method with storms of one-year return interval as the basis for design

SYSTEM DESCRIPTION

The Conventional Treatment System consists, in part, of a central treatment basin located at the head of Wilde Lake and the necessary facilities to collect and pump the runoff water to the treatment basin. As shown on Figure 46, the Conventional Treatment System is divided by natural boundaries into two parts. "Area A" consists of 954 acres of the watershed from which water flows by gravity to the treatment basin.

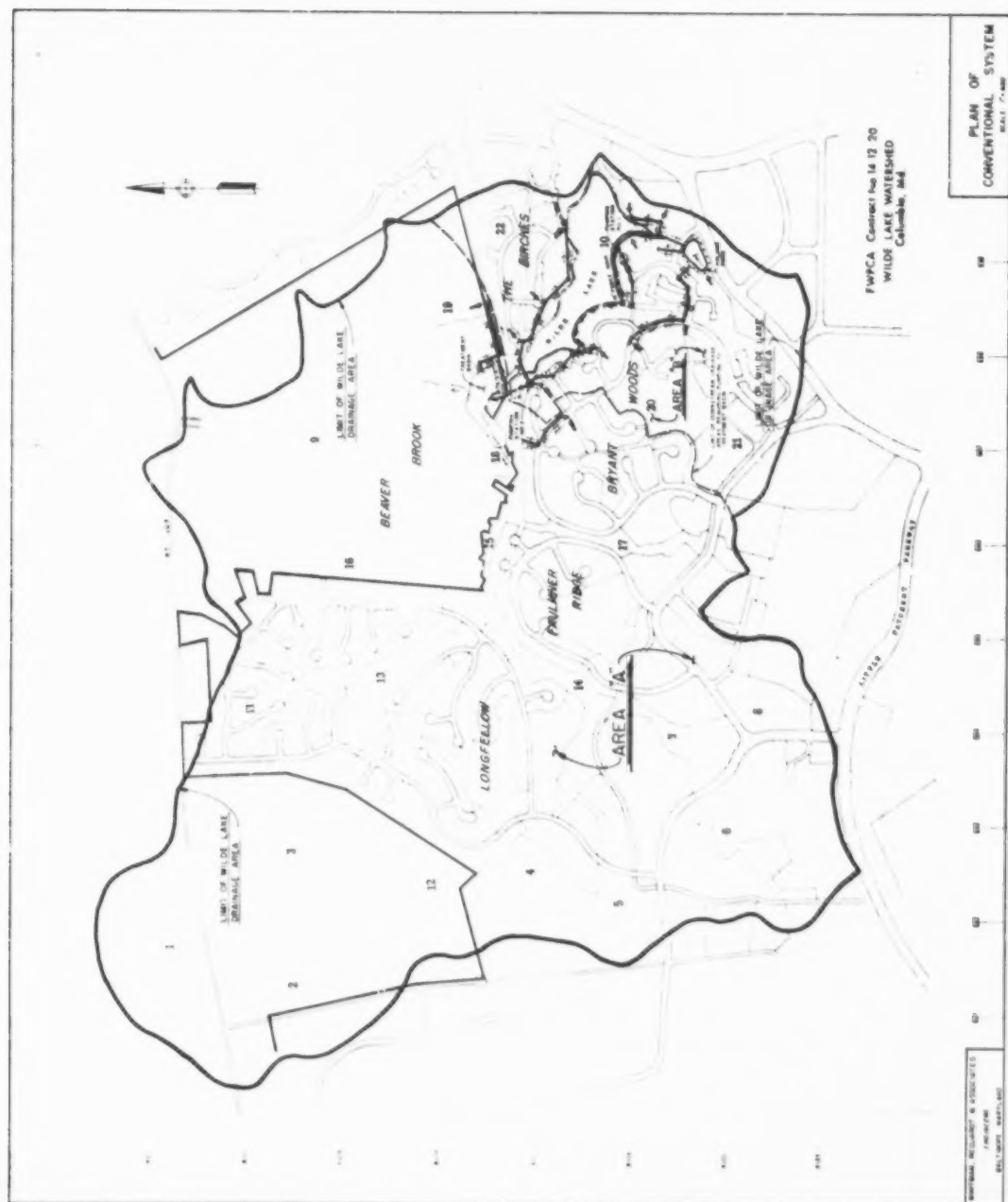


Figure 46. Plan of Conventional Treatment System

"Area B" consists of an area of 165 acres from which runoff must be collected and pumped to the treatment basin. The components comprising the Conventional Treatment System are as follows:

1. Holding Pond. This pond is located at the east side of "Area B" and has a capacity of 1,840,000 gallons. The pond receives the runoff from the major portions of sub-watersheds 10, 20, and 21 from a gravity interceptor on the west side of the pond. The holding pond provides storage capacity in order to avoid pumping runoff water at storm rates.
2. Pumping Station No. 1. This station is located on the south side of Wilde Lake and pumps the water from the holding pond to Pumping Station No. 2 for transfer to the treatment basin. This station also receives flow from gravity interceptors which collect runoff from the watershed adjacent to the south side of Wilde Lake. This station has a pumping capacity of 2.5 MGD.
3. Gravity Interceptors. The northern-most gravity interceptor collects the runoff from the majority of sub-watersheds 9, 16, and 19, and is located to permit this water to flow by gravity directly to the treatment basin. The gravity interceptor located adjacent to the north side of Wilde Lake collects the runoff from the lower portions of the sub-watersheds, which flows by gravity to the wet well of Pumping Station No. 2.

A gravity interceptor located at the west of the head of Wilde Lake intercepts the majority of the flow from sub-watershed 17 in order for the water to flow by gravity to the treatment basin. A second gravity interceptor located adjacent to the southwest side of Wilde Lake collects the runoff from the lower portion of the watershed which flows by gravity to Pumping Station No. 2.

4. Pumping Station No. 2. This station receives runoff from the two gravity interceptors located adjacent to the north and southwest side of Wilde Lake. This station also transfers the waste from the force main from Pumping Station No. 1 to the treatment basin. Pumping Station No. 2 uses dual 5 MGD pumps for a maximum capacity of 10 MGD.
5. Treatment Basin. The treatment basin is located at the head of Wilde Lake as shown on Figure 46. Figure 47 is a more detailed drawing of the treatment basin. The treatment basin consists of an earth and concrete dam and an excavated basin sized to retain a one-year storm. The treatment basin provides retention of the storm water runoff to allow sedimentation of suspended solids and incorporates a rock dam and gravel filter for filtration of water. Chlorination equipment is provided to disinfect the effluent and is designed

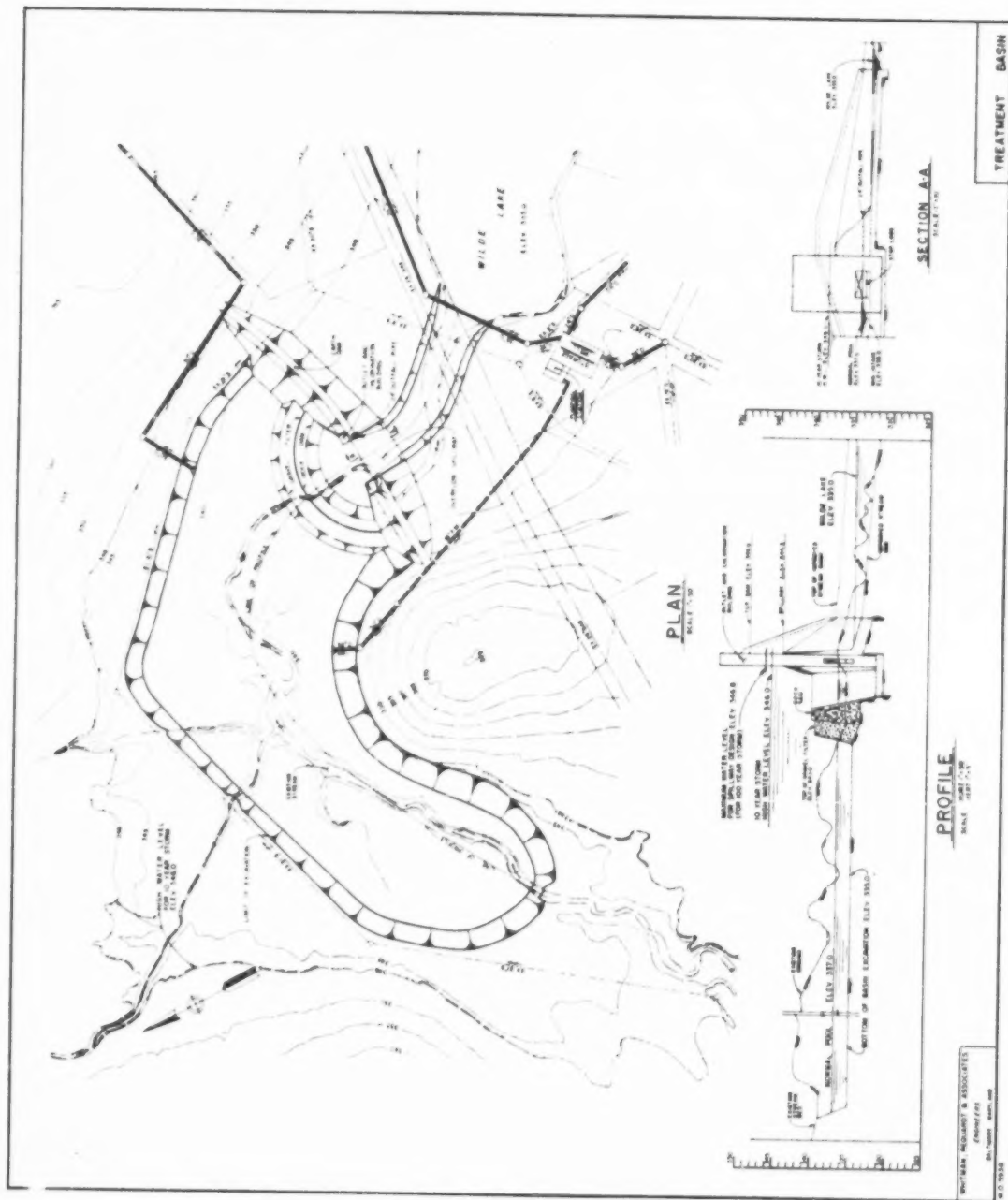


Figure 47. Conventional System Treatment Basin

to provide variable dosages to meet the range of overflow rates. The normal pool elevation is maintained by stop logs in the outlet structure and the pool may be drained for silt removal and other purposes by removal of the logs. The principal features of the treatment basin are as follows:

Dam height	15 feet from existing stream bed to top of dam elevation 349.0
Spillway length	60 feet
Spillway overflow elevation	344.5 feet
One-year storm maximum water level	344.0 feet
Normal pool storage	2.6 MG
Maximum storm storage	12.4 MG from normal pool to spillway overflow
High water level	346.0 (10-year storm)
High water volume	16.6 MG from normal pool to high water level
Area at high water level	424,000 square feet (9.75 acres)
Maximum water level for dam design	346.8 (100-year storm)

The outlet from the treatment basin is a 24-inch conduit which will drain the basin in 19 hours at an average flow of 20 MGD varying from 27.5 MGD at the high water level elevation to 0.5 MGD at the low pool elevation.

PERFORMANCE CHARACTERISTICS

The Conventional Treatment System is designed to collect and transfer storm water runoff to the treatment basin at the runoff rates and quantities of a one-year storm. The system is designed to drain the holding pond and the treatment basin to the minimum levels within a 24-hour period in order to minimize the probability that these storage basins will not be sufficiently empty to receive subsequent storms. The treatment basin is designed to meet the State of Maryland requirements for a Class 3 impoundment and the dam and spillway design is based on a 100-year storm. However, at storms of greater than one-year return interval, the capacity of gravity interceptors and the pumping stations may be exceeded and a portion of the runoff will overflow directly to

Wilde Lake. All of the runoff flowing directly to the treatment basin would be retained for some period.

The treatment provided the storm water with the Conventional Treatment System consists of sedimentation, gravel filtration, and chlorination. The removal of suspended solids and resulting reduction of turbidity in storm water by natural sedimentation is a function of the water surface area and overflow rate of the reservoir. Table 48 contains a listing of the size particles that would be removed with the treatment basin used for the Conventional Treatment System.

TABLE 48. PARTICLE REMOVAL WITH
CONVENTIONAL TREATMENT BASIN

Water Elevation	Effluent Q (MGD)	Water Surface Area (square ft)	Overflow Rate (gal/day/sf)	Size of Particles Removed (Microns)
337.0	10.0	181,000	55	7.5
338.0	14.0	186,000	75	8.8
340.0	19.5	196,000	100	10.1
342.0	24.0	219,000	110	10.6
344.5	28.0	320,000	89	9.5
346.0	278.0	424,000	660	26.0

Table 48 shows that the treatment basin will remove silt particles (approximately 10 microns) for overflow rates up to the spillway (elevation 344.5 feet) which includes the one-year storm (elevation 344.0 feet). However, it is considered that Class "C" water will require a greater reduction of turbidity and settlement of particles smaller than 10 microns.

In order to provide for treatment to a quality comparable to Class "C," the treatment basin incorporates a rock dam backed by a gravel filter through which storms of less than 5,000,000 gallons of runoff would pass, as would at least this amount of runoff from storms greater than 5,000,000 gallons. Storms of 5,000,000 gallons magnitude are exceeded six percent of the time. In terms of quantity of runoff, the 5,000,000 gallon filtration capability would represent about 60 percent of the total.

A storm of 18,600,000 gallons per day on the entire drainage will utilize 12,400,000 of the storage capacity of the basin without overtopping the spillway. For the five years of record, storms of 18,600,000 gallons per day were exceeded 0.25 percent of the time, which represents two to three percent of the total flow.

DESIGN AND CONSTRUCTION COSTS

As part of the conceptual design of the Conventional Treatment System, an estimate of the design and construction costs was prepared. Table 49 contains a summary of this estimate.

TABLE 49. DESIGN AND CONSTRUCTION COSTS -
CONVENTIONAL TREATMENT SYSTEM

Construction Costs	
Holding Pond	\$ 24,000
Pumping Station No. 1	117,000
Force Main	30,000
Interceptor System	205,000
Pumping Station No. 2	300,000
Treatment Basin	263,000
Subtotal	\$ 939,000
Design, Field Engineering, & Inspection	187,800
Contingencies and Escalation	187,800
TOTAL	\$1,314,600

The \$1,314,600 construction cost for the Conventional Treatment System translates to a daily cost of \$198.50 for the system using the same amortization rate previously used with the Local Storage, Treatment, and Reuse System.

OPERATION AND MAINTENANCE COSTS

The operating and maintenance costs for the Conventional Treatment System were estimated at \$25,000 per year which translates to \$68.49 per day.

SECTION XVII

ECONOMIC COMPARISON

Table 50 is an economic comparison of the four systems presented in Sections XIII, XIV, XV, and XVI. To more thoroughly analyze these results, a benefit/cost evaluation was made of the various system alternatives.

POTABLE REUSE

Capital costs associated with the optimum potable reuse system presented in Section XIII were \$1,445,200. When these costs are amortized over the life of the facilities, capital costs are found to be equivalent to uniform costs equaling \$207.21 per day. Operating and maintenance costs are estimated at \$194.10 per day. The sum of capital, operating, and maintenance costs is the total system cost and can be found to equal \$401.31 per day over the estimated life of the facilities. Offsetting these costs are tangible benefits from two sources. The treated water which is produced for reuse has been evaluated at \$0.45 per thousand gallons and totals \$278.00 per day in the case of potable reuse. The total system cost of the conventional storm water treatment system was computed as \$256.99 per day (based on capital costs of \$1,314,500 and annual operating and maintenance costs of \$25,000). Taking the imputed value of water pollution control as the total system cost of the best available alternative, total benefits can be computed as the sum of the value of reused water and the cost of conventional treatment, or \$534.99 per day. These computations are summarized as Case 1 on Table 51.

This computation of benefits and costs can be used to evaluate a benefit/cost ratio of 1.26 or a daily net benefit of \$105.88 depending on the purpose of the analysis. Several other factors bearing on this analysis should be considered in the interest of a comprehensive evaluation. The benefit/cost summary just outlined is appropriate to a water resource project having a variety of purposes and potential benefits. Two of the principal benefits, water supply and water pollution control, have been quantified and compared to total system costs. Other intangible benefits are discussed in later sections of this chapter and must be considered as part of the evaluation. The costs and benefits presented above can be organized somewhat differently in the case of projects intended principally as water supply facilities or as water pollution control facilities. Table 51 includes analyses of this type and lists the cost of potable water after full credit is taken for the water pollution control aspect of the system, as well as the cost of water pollution control after full credit is taken for the value of the potable water. Both of these costs are shown in units which permit ready comparison to other cost indices.

Critical to the calculation of benefit/cost ratios is the method used to evaluate benefits. The benefits noted above and on Table 51 are computed consistent with the assumptions described in other sections of this report. The sensitivity of the results to these assumptions can be investigated

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TABLE 50. ECONOMIC COMPARISON OF POLLUTION CONTROL SYSTEMS

	Capital Costs (\$)	Amortized Capital Costs (\$/day)	Average O&M Costs (\$/day)	Value of Reuse Water (\$/day)	Net Cost of System	
					(\$/day)	(\$/acre-year) (\$/DU-year)
Local Pollution Control System	829,800	118.99	54.79		173.78	55.25 16.88
Local Potable Reuse System	1,445,200	207.21	194.10	250.20	151.11	48.05 14.68
Local Sub-Potable Reuse System	2,598,000	151.51	372.64	206.94	317.21	100.85 30.82
Conventional Treatment System	1,314,600	189.50	68.49		256.99	81.71 24.96

TABLE 51. BENEFIT/COST ANALYSIS OF WATER SUPPLY
AND POLLUTION CONTROL ALTERNATIVES

	Case 1	Case 2	Case 3	Case 4
Costs				
Capital Cost of System:				
Amortized Capital Cost (\$/Day)	\$ 1,443,200.00	\$ 1,443,200.00	\$ 2,340,600.00	\$ 2,500,800.00
Operating and Maintenance Cost (\$/Year)	70,043.00	70,043.00	372.64	372.64
Operating and Maintenance Cost (\$/Day)	194.10	194.10	35.00	35.00
Total System Cost (\$/Day)	401.31	401.31	151.31	151.31
			324.15	324.15
Benefits				
Water Reuse				
Quantity of Reused Water (Gallons/Year)	202,942,000.00	202,942,000.00	167,897,000.00	167,849,000.00
Value of Reused Water (\$/Year) (\$0.45/1000 Gallons)	91,324.00	91,324.00	75,527.00	75,512.00
Value of Reused Water (\$/Day)	250.20	250.20	206.94	206.94
Water Pollution Control				
Capital Cost of Conventional System:				
Amortized Capital Cost (\$/Day)	1,314,600.00	829,800.00	1,314,600.00	829,800.00
Operating and Maintenance Cost (\$/Year)	25,000.00	116.90	23,000.00	116.90
Operating and Maintenance Cost (\$/Day)	68.49	32.00	63.49	32.00
Total System Cost (\$/Day)	256.99	54.79	173.76	54.79
Total Benefits (\$/Day)	507.19	423.96	483.93	380.72
Multipurpose Water Resource Project				
Net Benefit (\$/Day)	105.66	22.67	-60.22	-143.43
Benefit/Cost Ratio	1.26	1.06	0.80	0.73
Water Supply Project				
Total Cost Less Water Pollution Benefit (\$/Year)	32,600.00	83,030.00	97,310.00	127,890.00
Net Cost of Treated Water (\$/1000 Gallons)	0.26	0.41	0.36	0.76
Water Pollution Control Project				
Total Cost Less Water Supply Benefit (\$/Year)	33,160.00	35,160.00	113,730.00	113,700.00
Net Cost of Pollution Control (\$/1000 Gallons)	46.05	43.05	100.85	100.85
	14.60	14.60	30.82	30.82

\$/Day Dewatering Unit

Case 1. Potable Reuse System vs. Conventional Storm Water Treatment

Case 2. Potable Reuse System vs. Local Storage Without Reuse

Case 3. Sub-Potable Reuse System vs. Conventional Storm Water Treatment

Case 4. Sub-Potable Reuse System vs. Local Storage Without Reuse

rather easily. The value of reused water, for example, has been taken at \$0.45 per thousand gallons. Should this water be purchased by the local water utility and distributed and sold by them, the value of the treated water should conform to the cost of the water utility of an alternative supply. In the case of the Howard County Metropolitan Commission, this would be the incremental cost of water purchased from Baltimore City as well as other incremental costs associated with pumping, storing, and conveying that water to the Wilde Lake watershed. An exact determination of this cost would require a thorough study of the financial structure of the Metropolitan Commission. Another consideration might be the effect of higher water values, such as those existing in other areas of the country, on the economic evaluation. A comparison of this type is of limited value, since the system cost figures refer to a specific installation at Columbia, Maryland, and costs might vary widely in different locations. Case 1 on Figure 48 shows the relationship of the Net Benefit measure developed in Table 51 to the value of the reused water. It can be seen that the net benefits are zero, that is, the system can be considered break-even in an economic sense when the value of water is approximately \$0.26 per thousand gallons. When the value is set higher than this level, positive net benefits result, and when it is lower, a net cost develops which must be compared to the nonquantified secondary benefits described in later sections.

Although the conventional storm water treatment system developed by Whitman, Requardt and Associates can be said to be fairly representative of the best available method of achieving the water quality objective using current technology, the sensitivity of the conclusions to pollution control costs associated with advanced technology should be investigated. The local storage concept itself offers an advanced method of water pollution control when optimized for that purpose alone, without reuse. This type of system is described in Section XVI and will be compared to the conventional approach later in this chapter. Using the costs of the local storage system without reuse as the imputed value of water pollution control also permits convenient examination of the economics of reuse separately from the storm water control and treatment aspects of the system. Case 2 on Table 51 summarizes economic analyses of the potable reuse system as a multipurpose project, in each case using the costs of the local storage system without reuse as the imputed value of water pollution control. The lower value assigned to the water pollution control function results in lower net benefits in the multipurpose analysis and a higher incremental cost of treated water. The analysis of the system as a water pollution control project is, of course, unchanged. Case 2 on Figure 48 shows the effect of the value of treated water on the net benefit function when the local storage system is used as the source of water pollution control costs.

SUB-POTABLE REUSE

The optimum system for sub-potable reuse is described in Section XIV. The cost of this system totals \$2,598,600, of which \$1,562,700 represents distribution system costs. These costs are amortized over the life of the facilities and added to the operating and maintenance costs, resulting in

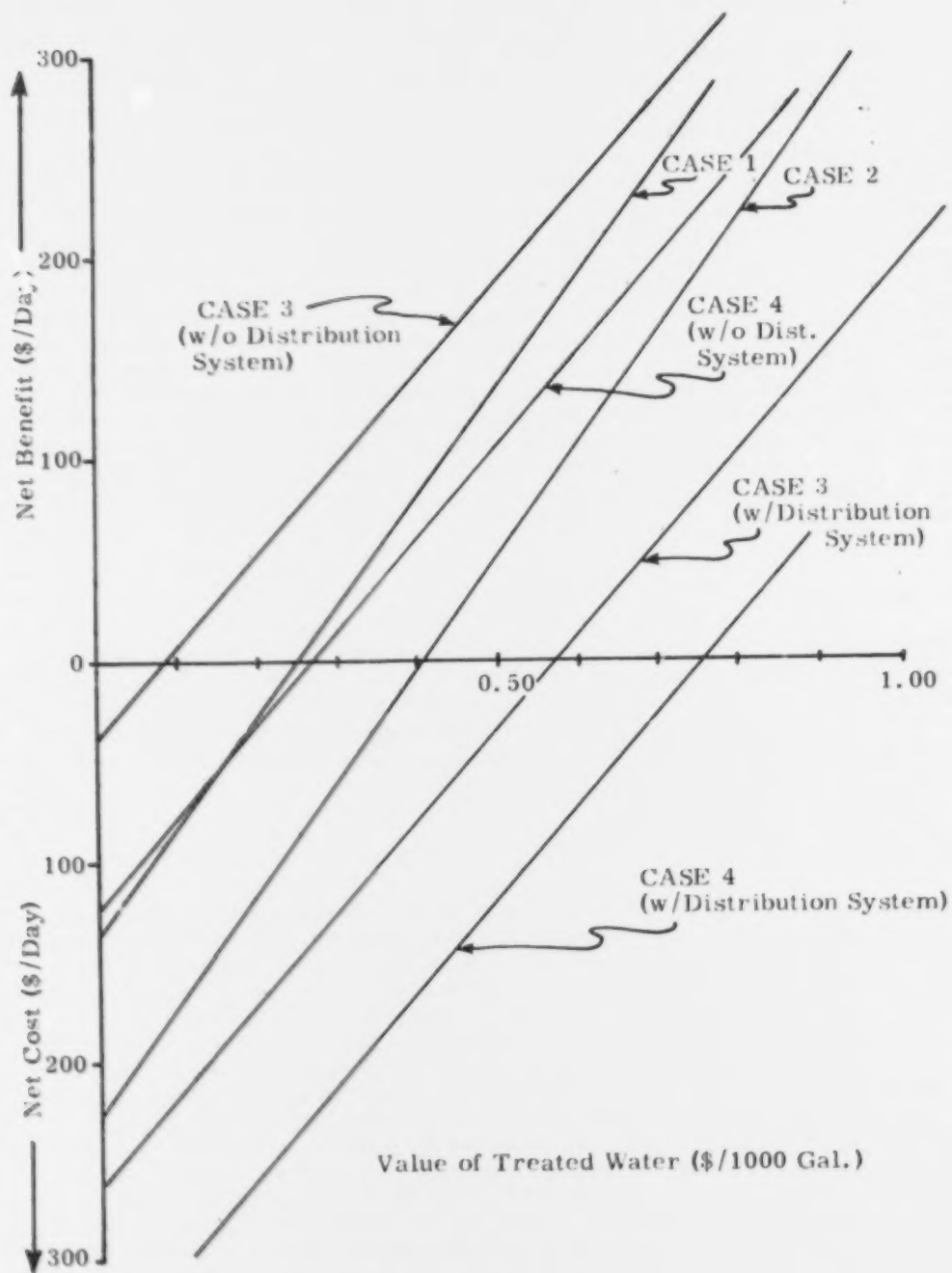


Figure 48. Net Benefits vs. Value of Treated Water

a total system cost of \$524.15 per day. Further details appear as Case 3 on Table 51. Total benefits are calculated at \$463.93 per day, producing a negative net benefit or net cost of \$60.22 per day. The sub-potable reuse system then, when considered as a multipurpose project, fails to generate tangible benefits equal to the costs.

Other comparisons were carried out as described in the previous section to investigate the effect of water supply and water pollution control considerations. The results of the analysis as a water supply project are further illustrated by Case 3 on Figure 48, where the effect of various values of treated water is plotted. Figure 48 also illustrates the influence of the cost of the distribution system on the overall results. It can be seen that treated water could be priced at \$0.10 per thousand gallons at the treatment plant (the zero cost intercept of plot "without distribution system") but that the cost of providing the distribution system increases the break-even price to \$0.58 per thousand gallons. Recomputation of benefits based on the use of the local storage system to establish the value of water pollution control leads to Case 4. It can now be seen that the portion of the total system cost allocable to water supply is substantially higher, but that the apparent cost of water pollution control remains at \$30.82 per dwelling unit.

NO REUSE

Costs for the local storage system without reuse have been presented in previous sections as an alternative measure of the value of water pollution control. Total system cost has been given as \$173.78 per day, compared to \$256.99 per day quoted for the conventional system. In addition, the local storage system offers the secondary, unquantified benefits described later in Section XVIII.

SUMMARY

The economic comparison of the three local storage systems and the conventional storm water treatment system has been conducted in several different ways. When the local storage systems are treated as multipurpose water resource projects, only the potable reuse system yields a benefit/cost ratio greater than unity, regardless of the base employed for evaluation of water pollution control benefits. The ratio was as high as 1.26 when the conventional system was used as the base and fell to 1.06 with the local pollution control system as the base. Investigation of the value placed on treated water revealed that all values greater than \$0.26 per thousand gallons resulted in positive net benefits when the conventional system was employed as a base. Use of the local pollution control system as a base increased the break-even point to \$0.41 per thousand gallons. When the system was analyzed as a water pollution control project, the net cost of pollution control amounted to \$55,160 per year, or \$14.68 per dwelling unit-year. This can be compared to the cost of pollution control by conventional means of \$24.97 per dwelling

unit-year and \$16.88 per dwelling unit-year by the local storage system without reuse. Analyzed on another basis, it can be found that if the system is installed as a pollution control system and the necessary facilities for potable water reuse are added, the additional cost involved in treating and distributing the potable water will total \$0.41 per thousand gallons of water distributed.

The sub-potable reuse system exhibits benefit/cost ratios as low as 0.73 and is characterized by a moderately high net cost even after full credit is taken for all quantified benefits. The cost of treated water is higher than the cost of water from the present alternative source when the system is analyzed as a water supply project. Furthermore, when analysis is based on water pollution control considerations, the cost of pollution control is higher than any of the alternatives. The excessive cost of the sub-potable system can be seen to be wholly caused by the cost of the sub-potable distribution system. The construction, operation, and maintenance of this facility alone accounts for an additional cost of approximately \$0.48 per thousand gallons of treated water which is more than the total incremental cost of the alternative supply of water.

Operation of the local storage system without reuse as a water pollution control method was compared to the conventional system and found to result in a net benefit of \$83.21 per day, somewhat less than the benefit associated with the operation of a potable reuse system which yields \$105.88 per day.

SECTION XVIII

SECONDARY BENEFITS

The primary objectives and benefits of the local storage, treatment, and reuse concept have already been discussed. Although the system was designed for optimum realization of these primary benefits, other benefits, both tangible and intangible, do result from the installation of such a system in an urban watershed. Although these benefits may be considered secondary, they are nevertheless important and might be as economically significant in many watersheds as pollution abatement and reuse.

STORM WATER MANAGEMENT AND FLOOD CONTROL

As any area begins to urbanize, the hydrology of the watershed alters drastically. With an increase in imperviousness and improved channels, the runoff from a given storm increases both in volume and peak flow. The previously derived regression model and hydrograph analysis clearly indicate these effects. Such increases generally result in greatly raising the potential for flooding the low areas adjoining the waterways. Floodplains which, in a natural state, were subjected to inundation once every few years might become flooded several times each year.

To indicate the seriousness of this problem, reference can be made to Figure 5 which shows that a natural area which is developed to an imperviousness of 0.35 will yield over 400 percent more runoff from an inch of rainfall. The effects on the peak flow may be just as serious, with 100 percent to 300 percent increases, depending upon the configuration of the drainage basin.

In areas where the floodplain is developed, flooding resulting from urbanization can cause severe economic losses. Even in those areas where the floodplain is left as dedicated open space, the use potential decreases as the probability of flooding increases.

A system of small reservoirs located along the stream reaches within the watershed, as previously described, has the ability to significantly reduce the frequency of flooding. This has been demonstrated in the past with large reservoir systems, and the principles underlying these flood control projects hold true for the local systems discussed in this report.

As a means of graphically portraying the flood control potential of these reservoirs, an analysis was made of the operation of two small storage reservoirs in series during a large storm. For simplicity, the watersheds supplying these reservoirs are hypothetical, with approximately rectangular shapes. It was assumed that initially the watersheds had an imperviousness of 0.1 and, using the hydrograph analysis and stream routing equations, the runoff hydrographs at the outflow points in each basin were calculated for a five-year storm.

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A development of the basins was then assumed to take place with the imperviousness of the upper basin increasing to 0.5 and that of the lower basin to 0.3. This development also altered the lag time in each basin, reducing it from 12 minutes to four minutes in the upper basin and from 24 minutes to 12 minutes in the lower basin. Two more calculations of the runoff hydrographs were performed assuming first that no reservoirs existed, and then that reservoirs were installed immediately above the outflow point in each watershed. The three basin configurations used for analysis are shown in Figure 49 with the size of the areas and reservoirs indicated.

It was necessary to select an initial water level in the reservoirs since their ability to dampen flood peaks is dependent upon the available storage. To accomplish this, a frequency distribution of water levels in one of the proposed Wilde Lake reservoirs was prepared. A probability level of 0.85 was chosen as a reasonable (and conservative) criterion for selection of a storage level. The frequency distribution indicated that for 85 percent of the time, during the wettest year, the reservoir was at one-third capacity or less.

Using this value for both reservoirs, hydrographs were obtained for the developed area without reservoirs, and were then routed through the reservoir storage. This yielded three different hydrographs at the two outflow points: one from the natural area, one from the developed area with no reservoirs, and one from a reservoir system.

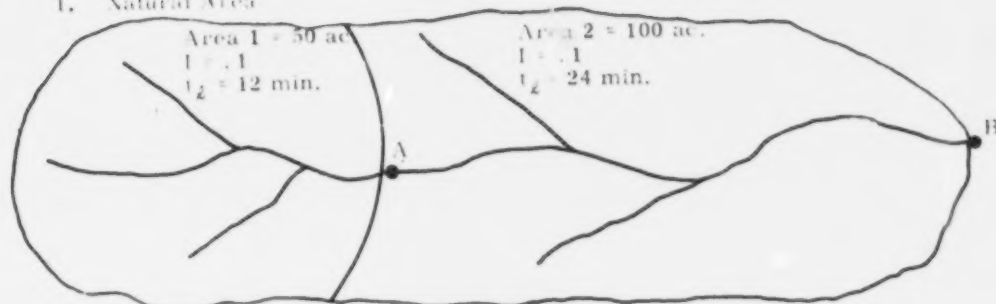
The results of this analysis are shown in Figures 50 and 51 which are plots of the runoff hydrographs at points A and B, the outflows of the two basins. It can be seen that at point A the development increased the peak flow by 112 percent with no reservoirs and actually reduced it to a level below the natural peak with a reservoir. At point B the increase was 110 percent with no reservoir and only nine percent with the reservoir.

This analysis clearly indicates the ability of the local storage reservoirs to dampen the runoff peak for even very large storms. The savings in economic losses due to flooding in urban areas can be an important consideration in any analysis of the desirability of such a reservoir system.

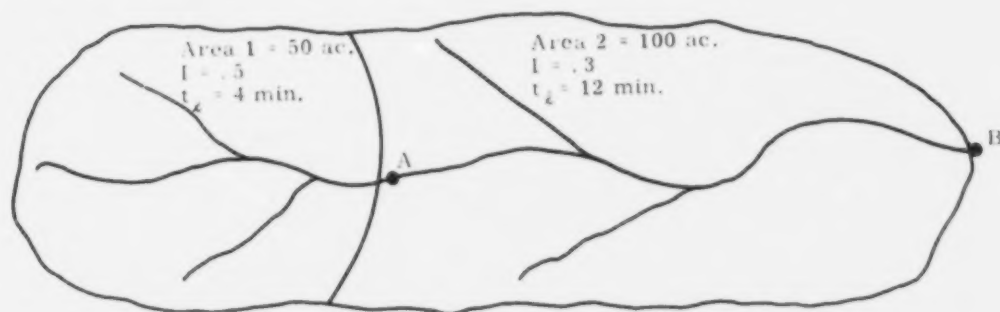
EROSION CONTROL

Any area, whether in a natural condition or undergoing urbanization, is subject to erosion. This process is entirely a natural one, resulting from the action of both rainfall and runoff on the ground surface and stream channels. It may, however, be tremendously accelerated by man-induced changes in the hydrology of an area. Such accelerations are well documented, with typical sediment yields from areas under construction or already developed 10 to 1000 times those from the natural area.

1. Natural Area



2. Developed Area without Reservoirs



3. Developed Area with Reservoirs

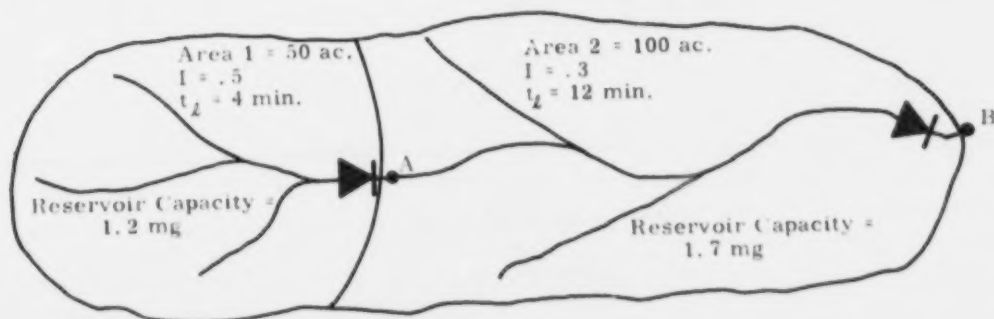


Figure 49. Basins Used for Hydrograph Damping Investigation

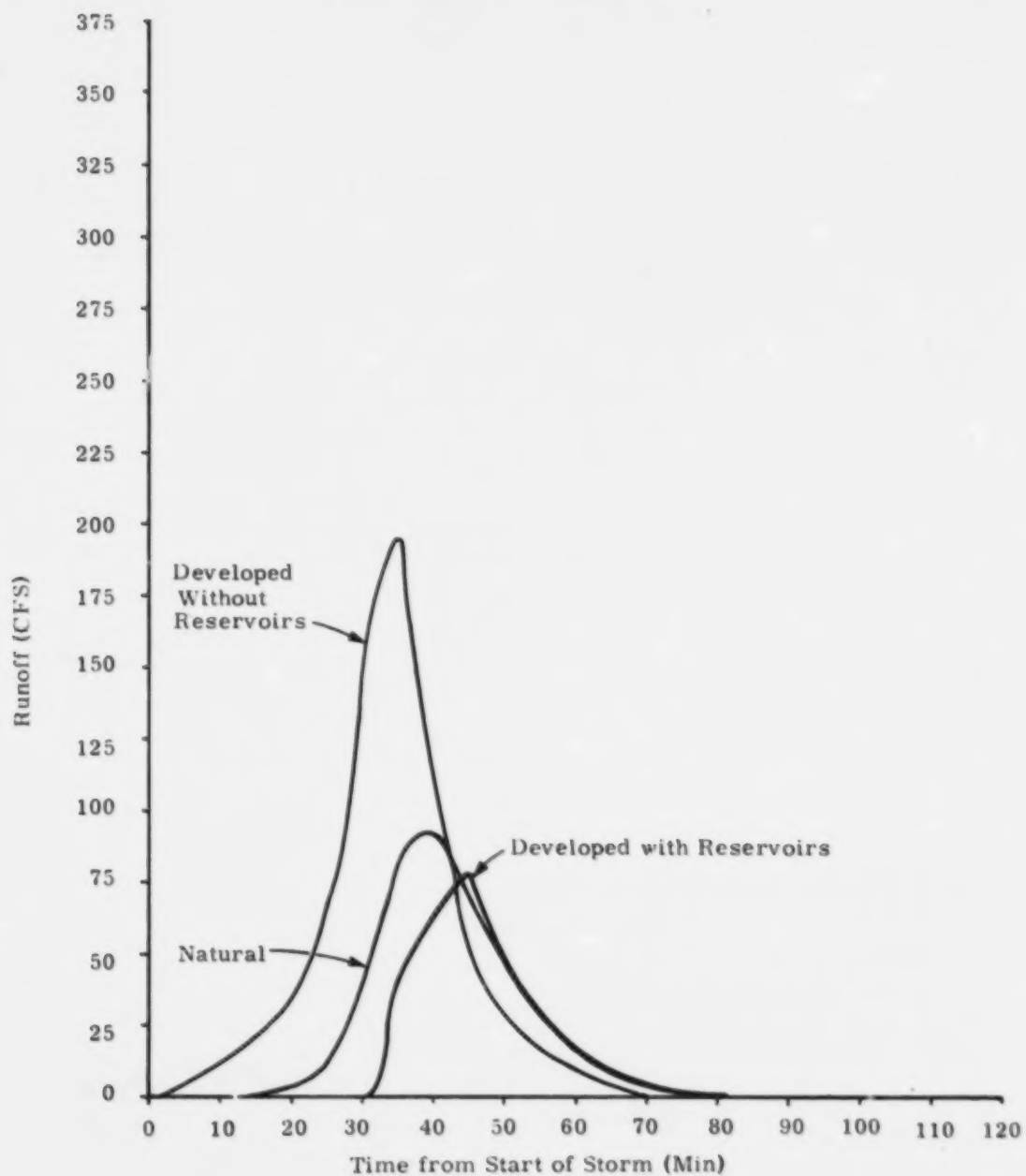


Figure 50. Runoff at Point A

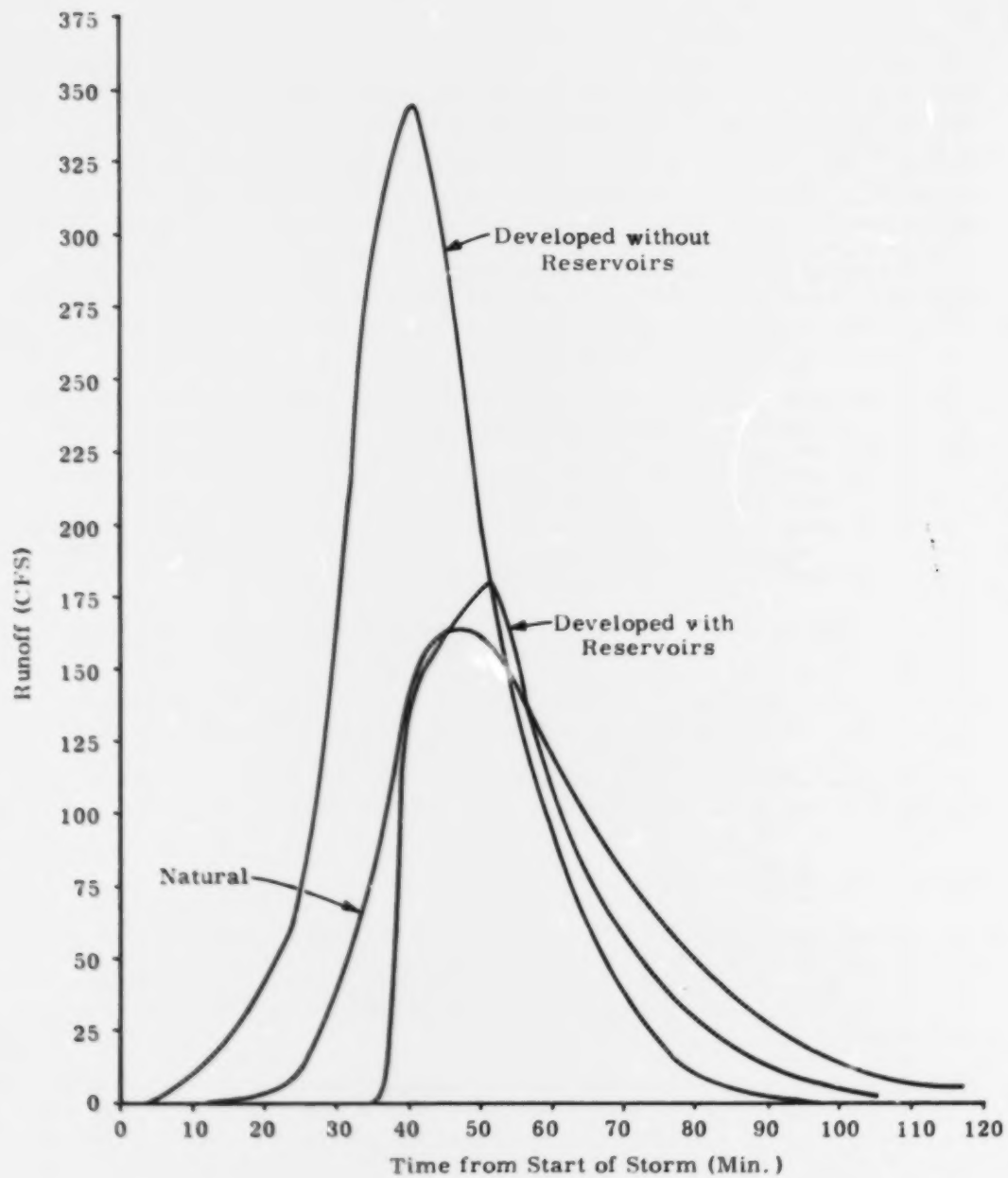


Figure 51. Runoff at Point B

Although much of this increase in sediment and erosion can be attributed to the stripping of vegetation from the land during development, a still significant portion results from the increased runoff discussed previously. The erosion of either land surface or stream channels is a function of both the kinetic energy per unit volume and the total volume of the runoff. Since kinetic energy is proportional to the velocity squared, a large change in velocity or flow rate will have an even greater effect on the kinetic energy and erosion potential of the water. Velocity, flow, and volume increases are all significant during urbanization, and even after construction in a watershed has been terminated, the increased erosion of the stream valley continues.

There are many case histories of once natural streams becoming severely eroded when their watersheds are developed. The stream bottoms are cut deeper and the banks begin to become undercut and collapse. Deep gullies are formed in areas of the floodplain where there previously were small rivulets. Once a stream approaches this condition, there is generally public pressure to have the banks and bottom paved, completely destroying its original character. Improved channels further accelerate peak flows and create hydraulic and erosion problems downstream.

For the reasons discussed with respect to flood control, this condition does not have to occur in an area with reservoir-controlled runoff. The reduction of flows and high volumes of water by reservoir storage can help to retain the natural stream channels and floodplains. Although dampening hydrographs do not always lead to a proportionate decrease in downstream erosion (due to changes in flow regimen), the ability to control release rate and the diversion of a substantial quantity of water to reuse should result in significant reduction in channel erosion.

NUTRIENT REDUCTION

An increasing awareness of the problems of nutrient enrichment of lakes and estuaries is developing throughout the country. Although many causes of such enrichment are offered, there is general agreement that urban runoff can add a significant amount of nitrates and phosphates to a receiving body.

The operation of the treatment plants associated with the storage reservoirs cannot be expected to remove more than a token quantity of the nutrients present. By returning a sizable portion of the runoff to the watershed through the reuse system, the nutrients will eventually be transported out of the area through the sanitary sewers.

While this does not remove the nutrients from the water, it does have two beneficial effects: it prevents a large portion of the nutrients from entering the small urban lakes which are prone to eutrophication, and it transfers them to a large treatment plant where they may be eventually removed economically.

ECOLOGICAL PRESERVATION

As a watershed changes from rural to urban, there is generally a drastic alteration of the ecological characteristics of the streams and their floodplains due to the increased storm flows, high silt loads, erosion of natural banks, and pollution.

A typical history of a small stream situated in a watershed undergoing urbanization might be as follows:

During the stripping and excavating of the land, which takes place in the construction phases of development, the stream begins to become clogged with sediment. Bottom organisms found in the natural stream cannot live once the thick layer of silt has settled over the previously rocky bottom. Larger organisms, including the original fish population which were dependent upon these organisms, begin to disappear. As construction proceeds, larger areas of the watershed become impervious and the storm flows greatly increase. The former banks of the stream fail to hold the increased flow, and mud flats are deposited along the stream valley. Erosion increases greatly and the combination of erosion and mud deposition begins to destroy the natural vegetation along the stream banks and in the floodplain. Eventually this entire area changes in appearance with little of the natural flora or fauna remaining. Trash, debris, and pollutants carried by the stream help to spawn a new ecology, usually including rats, leeches, scavenger fish, and mosquitoes. The stream valley in this condition is shunned by the residents of the watershed.

Such a history, although typical, is not necessary. As was previously discussed, a series of reservoirs within the floodplains of the streams can abate many causes of the destruction of a natural ecology. The elimination of most sediment, the dampening of floods, and the reduction of erosion can assist in the maintenance of the original ecological conditions not only in the stream but in the surrounding floodplain as well. The effects of preserving this original appearance on the recreational potential of the area are discussed in the following section.

RECREATIONAL AND AESTHETIC CONSIDERATIONS

Water is an important part of our natural landscape and environment. Its value has been recognized and employed by planners and administrators in projects such as the Baltimore City Inner Harbor Development and the Columbia City Project in Maryland. The aesthetic and recreational value of water, whether in a natural or urban setting, cannot be overemphasized.

In the case of the local storage ponds, the setting would be primarily a natural one with as little excavation as possible and immediate replanting of natural vegetation. The choice and treatment of construction materials would become a part of, and possibly enhance, the local natural environment.

The ponds present the opportunity for development of both active and passive recreation areas adjacent to the water. Aside from creating a body of water, the development of the ponds produces clear flowing streams of good quality water, thus improving the entire floodplain. With the now decreased flood activity, the floodplain can be developed and treated as useful productive land.

Three possibilities of handling the pond and its floodplain are a completely natural setting, development of a passive recreational area, and development of an active recreational area.

In the completely natural setting, care is taken during construction to preserve the character of the area, leaving as much natural vegetation and rock formations as possible. After construction, the area is landscaped to resemble the surrounding environment. The spillway is masked with rock or vegetation and the entire area is allowed to be overtaken by the wild vegetation present. Figure 52 shows the appearance of a typical pond in a natural setting.

The development of the floodplain as a scenic walk and picnic area with seating facilities and lighting is another possibility. Incorporating the pond, spillway, and natural surroundings into a creative, stimulating design could prove quite valuable to the local community. Such a design concept is illustrated by Figure 53.

Simple structures could be developed in more populated areas to turn the pond and spillway area into a recreational playground facility. In the event that local conditions dictate fencing of the ponds, the development of these recreational facilities will produce a positive result by having the fencing contain a usable function rather than merely guarding the pond. One recreational design that might be considered for such an application is shown in Figure 54.



Figure 52. Local Storage Pond in Natural Setting



Figure 53. Local Storage Pond in Wooded Park Setting



Figure 54. Local Storage Pond in Recreation Area Setting

SECTION XIX

DEMONSTRATION PROGRAM

Upon completion of this study in 1968, the results were reviewed with the State of Maryland, the Environmental Protection Agency, Howard County officials, and the developers of Columbia, Maryland. It was generally agreed that the concept of providing retention of storm water for pollution control and as an alternative water supply had merit. However, it was the consensus that the control of sediment generated by construction and urban development was a greater pollution problem than post-development pollution control and that these aspects of the study should be given priority in any subsequent demonstration programs. During this period, the State of Maryland enacted House Bill No. 1151 for the control of grading, erosion, and sediment during construction and development throughout the state. In the interest of developing additional guidelines for the implementation of this law, the Department of Water Resources of the State of Maryland agreed to serve as the sponsor of the demonstration project, and Howard Research and Development Corporation and the Columbia Parks and Recreation Association agreed to contribute to the project. An application was made to the Environmental Protection Agency and the demonstration project was initiated in July 1970. The scope of this demonstration project, which is now in progress and scheduled for completion in late calendar year 1972, is described in this section.

OBJECTIVES

The objectives of the Environmental Protection Agency demonstration grant project 15030 FMZ as specified in the grant application and offer are as follows:

1. Evaluate the effectiveness and costs of advanced methods of erosion control in urban areas
2. Evaluate the effectiveness and costs of various methods for the transport, drying, conditioning, and disposal of sediment
3. Demonstrate urban storm water pollution control during construction and following development by the use of a local storage and treatment system
4. Collect and analyze data on the effects of urbanization and various control measures on hydrology, water quality, stream ecology, and channel hydraulics

In addition to the regular reports, an "Erosion and Sediment Control Manual for Urban Development" is to be prepared covering the erosion

and sediment control practices demonstrated under this project and other practices that can be used in urban development.

DEMONSTRATION SITE AND FACILITIES

The demonstration project is being conducted on a 200-acre watershed in the Village of Long Reach in the new city of Columbia, Maryland. At the beginning of the project in 1970, only the rough grading of access roads within the watershed had been completed. This rural agricultural area was selected for the demonstration project because it was scheduled for complete urbanization over the period of the demonstration project.

Upon the initiation of this project, an earth-filled dam was constructed at the lower end of the demonstration watershed to form a three-and-one-half acre lake to serve as a sediment retention pond during the course of the construction and the project. As discussed later, this pond is also being used to demonstrate storm water management techniques and sediment handling and removal methods.

Two sub-watersheds at the upper end of the demonstration area have been respectively designated as reference and experimental sub-watersheds. Gaging and sampling stations have been installed to monitor the flows and to collect samples of the storm water draining from each of the two sub-watersheds. Within the reference sub-watershed, the erosion and sediment controls are those applied by the developers in accordance with applicable county and state regulations. Within the experimental sub-watershed, special methods of erosion and sediment control are being applied and evaluated.

Two other gaging and sampling stations have been installed. One is located immediately above the storm retention pond to monitor flows and collect samples of storm water runoff from the entire demonstration watershed. The other station is located downstream of the storm retention pond. This station records the overflow from the pond and the flow over the emergency spillway during major storm events.

Three recording rain gages are installed on the watershed and the clocks are synchronized with those used in the gaging and sampling stations.

EROSION CONTROL

In the demonstration project, primary emphasis is being placed on erosion control practices on the basis that the most effective method of control is the retention of the soil and the elimination of sediment as a pollutant. For purposes of demonstration, evaluation, and definition of future applications, erosion control practices have been categorized on the following pages.

Long-Term Stabilization Using Vegetative Cover

Limitation of grading, excavation, and fill to retain maximum natural cover

Limitation of period of exposure or denudation, especially with regard to frequency of high intensity rainfall events

Scheduling of earth-moving operations to utilize germination and growing seasons, whenever possible

Design for and use of proper equipment in the preparation of soils and the planting of vegetative cover

Use of natural material, matting, and/or artificial methods to:

- Stabilize areas during the planting and development of vegetative cover

- Provide long-term root reinforcement for vegetative cover on critical slopes and drainage ways

Use of special ground cover, shrubs, or trees to supplement permanent grasses on critical areas

Use of turf to provide expeditious development of permanent ground cover

Use of vegetative cover to stabilize engineered and temporary erosion and drainage control structures

Establishment of vegetation with higher velocity tolerance in areas of concentrated overland flow

Long-Term Engineered Erosion and Drainage Controls

Chutes, flumes, and energy dissipators to convey runoff in critical areas where volume and/or velocity is beyond vegetative tolerances

Erosion checks and level spreaders for reduction of flow velocity

Structural stabilization using rubble, stone, or grouted riprap

Sediment traps that may be converted to beneficial use (recreation, wildlife habitat, etc.) after development and construction operations have been completed

Trickle channel and outfall protection using prefabricated or field-fabricated devices

Interim Erosion and Drainage Controls

Temporary diversion and interceptor dikes

Level spreaders

Existing naturally vegetated waterways

Temporary chutes, flumes, and downdrains

Bituminous concrete

Reinforced synthetic fabric

Chemical stabilizers

Temporary sedimentation basins

Temporary seeding of denuded areas

Temporary mulching of denuded areas

Straw and hay

Chemical soil stabilizers

Chemical soil flocculants

Wood fiber

Fiber glass

Temporary stabilization of denuded areas

Mats

Natural material
Synthetic material

Nets

Plastic
Fiber glass
Natural material

Turf

Temporary erosion checks

Weirs (log, stone, chemically stabilized aggregate)

Natural fibers

Plastic materials

Fiber glass products

Chemical products

Emergency Erosion and Drainage Controls

Diversion ditches and berms

Diversion with nylon bags filled with soil or sand

Interceptors built of nylon bags filled with soil or sand

Chutes and flumes of rigid or semirigid plastic material

Chutes and flumes of chemical sprays

Chutes and flumes of synthetic fiber

Level spreaders of synthetic material

Chemical soil stabilizers, blankets, and flocculants

Interceptors built of rock and gravel

As a part of this project, a large number of manufacturers and vendors of erosion control and landscaping materials and equipment are providing products for demonstration and evaluation purposes. These include: chemical soil stabilizers and mulches, blanketing products, fabric form for bank and stream stabilization, hydroseeders, plus other materials.

SEDIMENT HANDLING AND DISPOSAL

A survey was made of methods for the removal of deposited sediment from storm retention ponds early in the project. The removal methods considered included the use of conventional construction equipment, use of underwater roads with conventional equipment, draglines, underwater scoops, barge-mounted back hoes, and small dredges. Based on the equipment available at the time, the use of an underwater scoop operated off deadmen and the use of conventional front loaders were selected for demonstration.

An "engineered forebay" has been constructed at the upper end of the storm retention pond. This consists of an underwater dam and a flow spreader which form a widened section in the channel entering the pond. The forebay serves two purposes. First, it provides an area in which the heavier sediment can be trapped before entering the pond and an area in which this material can be removed by front loaders with minimal disruption of the area around the pond. Secondly, the forebay provides an area to which the deposited sediment from the pond proper can be transported by the underwater scoop for dewatering and subsequent removal.

Laboratory tests have been conducted on the dewatering characteristics of various types of sediment with and without chemical additives. Test beds are to be constructed to demonstrate selected techniques in the field.

For the demonstration, conventional dump trucks will be used to transport the sediment to the dewatering and disposal site. As necessary, plastic liners will be used to stop leakage from the trucks.

STORM WATER MANAGEMENT

The key features of the local storage and treatment concept of pollution control considered in the original system study are being demonstrated. Using the gaging and sampling stations located upstream and downstream of the storm retention pond, data are being developed on the effectiveness of this method of storm water management. Special tests are being conducted in which the level of the retention pond is lowered between storm events. The effects of the additional available storage capacity on the downstream hydrographs are being determined. This will also provide information on the need to provide bank stabilization on ponds operated with variable water levels.

The size of the storm retention pond makes it a relatively effective settling basin for most suspended solids. Accordingly, it is being operated without supplemental treatment to determine the extent to which pretreatment and treatment would be required to use water from storm retention basins as a supplemental water supply.

EFFECTS OF URBANIZATION

The data from the rain gages and the gaging stations are being analyzed to determine the extent to which storm water runoff is being changed by the installation of storm sewers and the urbanization of the area. Actual site and rainfall data are being used as input to the Environmental Protection Agency, Storm Water Management Model (49), and the calculated runoff will be compared to actual field data.

Aerial photographs of the demonstration watershed area are being acquired quarterly to document the status of construction and development

and other site data. This information is being retained to allow generation of input data for future runoff models for comparison with actual runoff. These aerial photographs are also being used to monitor changes in stream and other drainage channels in the area.

In addition to the aerial photographs, ground surveys are being made quarterly of the retention pond and the major stream flowing to the pond. Profiles are being prepared from previously established bench-marks to determine the extent of stream erosion and the deposition of the sediment in the stream and pond.

An inventory of the aquatic life existing in the area was established early in the project and periodic ecological surveys are being made to monitor the changes that have occurred in the stream and pond during development.

Extensive photographic coverage of the erosion controls, sediment removal operations, facility construction, etc., is being maintained, and slides are being accumulated to show sequential effects.

SECTION XX

ACKNOWLEDGEMENTS

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APPENDIX A

DERIVATION OF HYDROLOGY EQUATIONS

Section IV of this report, "Storm Water Hydrology," contains a discussion of the methods used to describe the storm flow characteristics of the Wilde Lake drainage area. This appendix contains the derivation of the equations used to modify the regression model for calculating the runoff for individual storms which are contained in Section IV. These derivations are as follows.

Alteration of the Regression Equation

Let Q_{IP} = runoff/unit area from Herring Run

Q_P = runoff/unit area from the pervious portion of Herring Run or Wilde Lake

Q_I = runoff/unit area from the impervious portion of Herring Run

$$Q_{IP} = BR_i + CR_i^2 + DR_i^3 \quad (\text{neglecting base flow})$$

$$Q_{IP} = Q_P(1-I) + Q_I I \quad (I = \text{Fraction Impervious})$$

$$Q_I = KXR_i$$

$$Q_P(1-I) + Q_I I = BR_i + CR_i^2 + DR_i^3$$

$$Q_P = \frac{1}{1-I} (BR_i + CR_i^2 + DR_i^3) - \frac{KXR_i I}{1-I}$$

$$Q_P = \frac{1}{1-I} \left[(B-KXI)R_i + CR_i^2 + DR_i^3 \right]$$

Let $Q_{I'}$ = runoff/unit area from the impervious portion of Wilde Lake

$Q_{I'P}$ = runoff/unit area from Wilde Lake

$$Q_{I'} = KXR_i$$

$$Q_{I'P} = Q_P(1-I') + Q_{I'} I'$$

$$Q_{I'P} = KXR_i I' + \left(\frac{1-I'}{1-I} \right) \left[(B-KXI)R_i + CR_i^2 + DR_i^3 \right]$$

Adding the base flow term:

$$Q_i = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right) \left[(B-KXI)R_i + CR_i^2 + DR_i^3 \right]$$

Derivation of the Expression Defining X

$$Q_I = \frac{(1+I)^{0.339} (R)^{1.75} (API)^{0.0741}}{D^{0.0973}} \quad (\text{Ref. 3})$$

where:

- I = percent imperviousness
- R = precipitation in inches
- API = antecedent precipitation index
- D = duration of storm, minutes
- Q_I = runoff volume in inches from an area with I percent imperviousness

Let Q_o = runoff for $I=0$, and let X = fraction of rainfall on impervious areas that actually runs off. Then:

$$Q_I = \frac{XRI}{100} + Q_o \left(\frac{100-I}{100} \right)$$

$$Q_I - Q_o = \frac{XRI}{100} - \frac{Q_o I}{100}$$

$$X = \frac{100}{RI} Q_I \left[1 - \frac{Q_o}{Q_I} \left(1 - \frac{I}{100} \right) \right]$$

$$\frac{Q_o}{Q_I} = \frac{1}{(1+I)^{0.339}}$$

$$X = \frac{Q_I}{R} \left[\frac{100}{I} + \left(\frac{I-100}{I(1+I)^{0.339}} \right) \right]$$

Derivation of the Equation Defining P

Let P = fractional increase in unit yield of the developed area over the natural area

$$P = \frac{Q_{I'P} - Q_P}{Q_P}$$

where:

$$Q_{I'P} = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right)(B - KXI)R_i + CR_i^2 + DR_i^3$$

$$Q_P = A + \left(\frac{1}{1-I}\right)(B - KXI)R_i + CR_i^2 + DR_i^3$$

$$\text{Let } [(B - KXI)R_i + CR_i^2 + DR_i^3] = Y$$

$$P = \frac{A + KXI'R_i + \left(\frac{1-I'}{1-I}\right)Y - A + \left(\frac{1}{1-I}\right)Y}{A + \left(\frac{1}{1-I}\right)Y}$$

$$P = \frac{I' \left[KXR_i - \left(\frac{1}{1-I}\right)Y \right]}{A + \left(\frac{1}{1-I}\right)Y}$$

Simplifying:

$$P = \frac{I' \left[KX - B - CR_i - DR_i^2 \right]}{\frac{A(1-I)}{R_i} - KXI + B + CR_i + DR_i^2}$$

APPENDIX B

FIELD GAGING AND SAMPLING

GAGING STATION RESULTS

A temporary stream gaging station was installed in one of the sub-watersheds on which little development had taken place. This gaging station had two primary objectives:

To provide streamflow data to verify the regression model which was used to calculate runoff

To provide water quality data to compare with the data provided by the literature search

The watershed that was selected for gaging was a 130-acre area in the upper end of the Wilde Lake drainage basin scheduled for development during the end of 1968. At the time of installation, several roads had been completed, giving the area an imperviousness of six percent. The undeveloped portion of the area consists of woods and a meadow with two main stream channels joining several hundred feet above the gage site (Figure B-1).

The instrumentation (Figure B-2) consisted of a compound weir located at the end of a long reach of a straight channel approximately six feet wide and two and one-half feet deep. When the weir plate was put into place, the water formed a distinct pond for a distance of 40 feet upstream.

Stage measurements of discharge over the weir were made using a stilling well, float, and Stevens A35 water level recorder. This recorder was located in an instrument box and was powered by a storage battery and power inverter. It provided a continuous record of water level with both good time and stage resolution.

The stage measurements were related to discharge over the weir by the use of an equation derived by model studies conducted by the Bureau of Reclamation. Field measurements were made of the flow by timing the rate of fill of known-volume containers, and these measurements closely agreed with the values calculated by the equation.

Approximately 40 feet upstream from the end of the weir pond, an automatic water sampler was installed. This took water samples at preset time intervals and was usually hand-started whenever rain was forecast. These samples were collected and returned to the laboratory for analysis.

A University of Maryland weather station located a short distance from the watershed provided total rainfall information to be used in the regression model.

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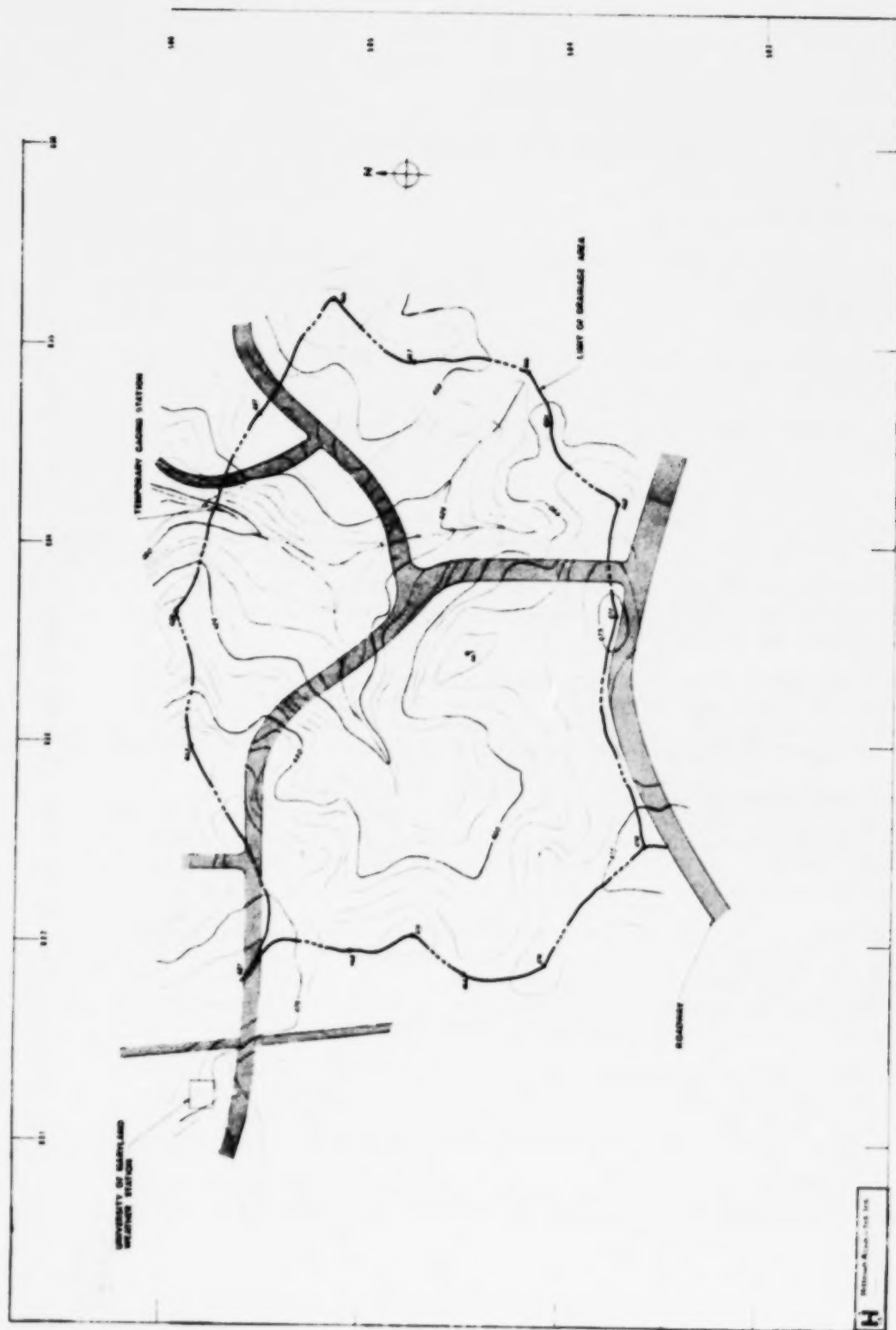


Figure B-1. Temporary Stream Gaging Station Drainage Area

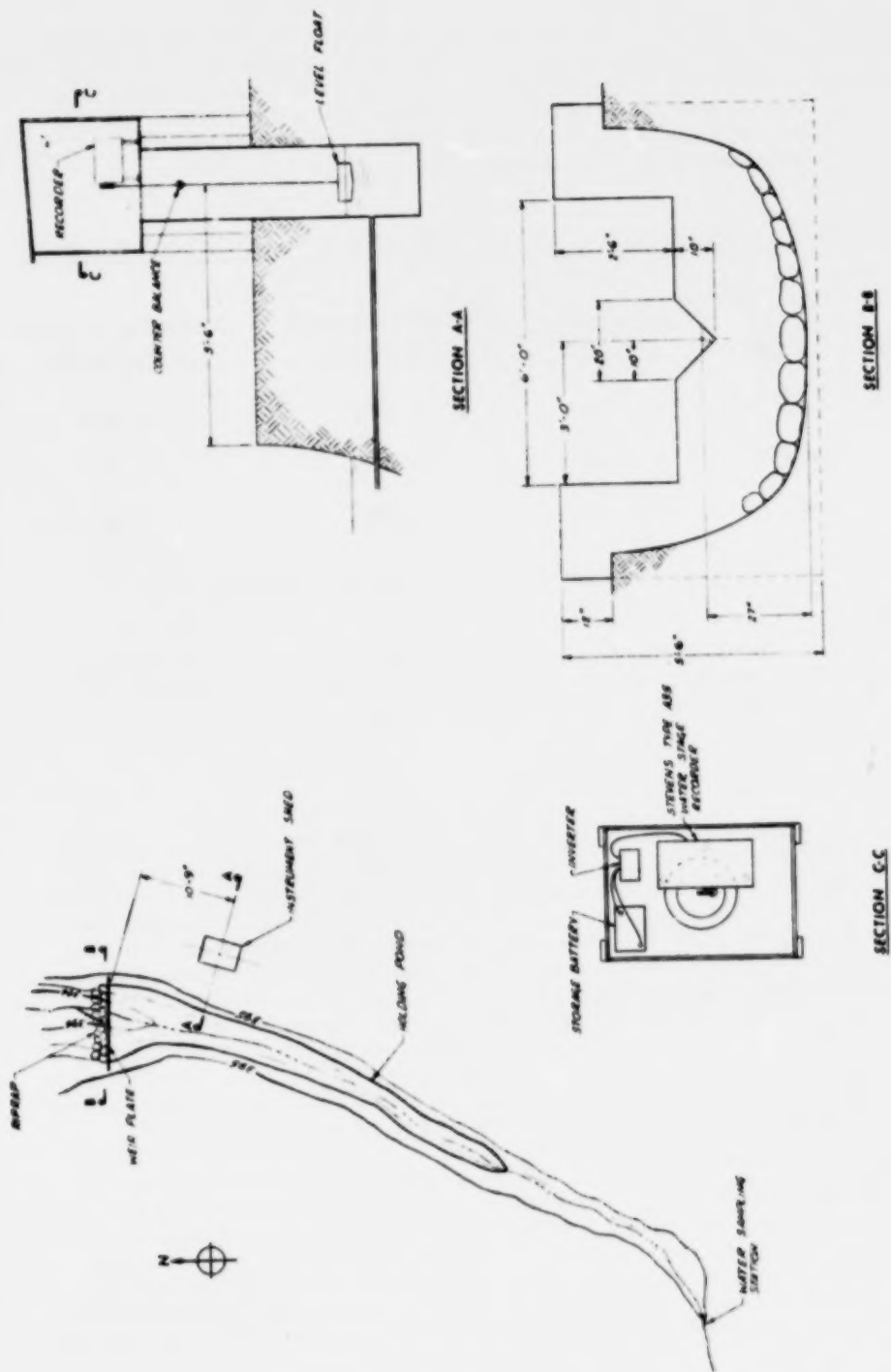


Figure B-2. Temporary Stream Gaging Station Site Plans

During the time of operation of the gaging station, three storms occurred from which runoff hydrographs were obtained. These three hydrographs are shown in Figures B-3 through B-5. From the hydrographs, the volume of runoff was determined and compared to the runoff volume calculated by regression model for the same amount of rainfall. Table B-1 gives the results of this comparison.

TABLE B-1. COMPARISON OF GAGED AND CALCULATED RUNOFF

<u>Storm</u>	<u>Date</u>	<u>Rainfall (inches)</u>	<u>Measured Runoff (cfs-days)</u>	<u>Calculated Runoff (cfs-days)</u>
1	6/26-6/27	0.79	0.242	0.352
2	6/27-6/28	0.66	0.236	0.254
3	7/19	1.16	0.70	0.60

Correlation between measured and calculated = 0.92

For the first and last storms, water samples were also collected. The results of the analysis of these samples, as well as the analysis of base flow, will be presented in a subsequent section.

FIELD SAMPLING RESULTS

In order to permit early verification of the expected pollutant concentrations listed in Table 10, a sampling point was established in the Wilde Lake Watershed in conjunction with the gaging station described above. An automatic sampler was used to collect runoff samples at one-hour intervals from a point immediately upstream from the weir pond. These samples were correlated with measured stream flow at the time of sampling and analyzed for various pollutants by Hittman Associates. The results of this analysis are presented as Tables B-2, B-3, and B-4. Tables B-2 and B-4 reflect water quality during two of the storms whose hydrographs were analyzed above and identified as Storms 1 and 3. Table B-3 lists data on water quality during a typical period of minimum flow, when no storm runoff was present.

All samples were excessively high in suspended and total solids, due to a high level of construction activity in the watershed during the period under study. A large portion of the floodplain itself had been denuded as a result of the construction of a sanitary sewer, and erosion was severe. For this reason, the results obtained cannot be treated as typical of this area. Furthermore, little urbanization has occurred on the gaged watershed at the present time. At the time of sampling,

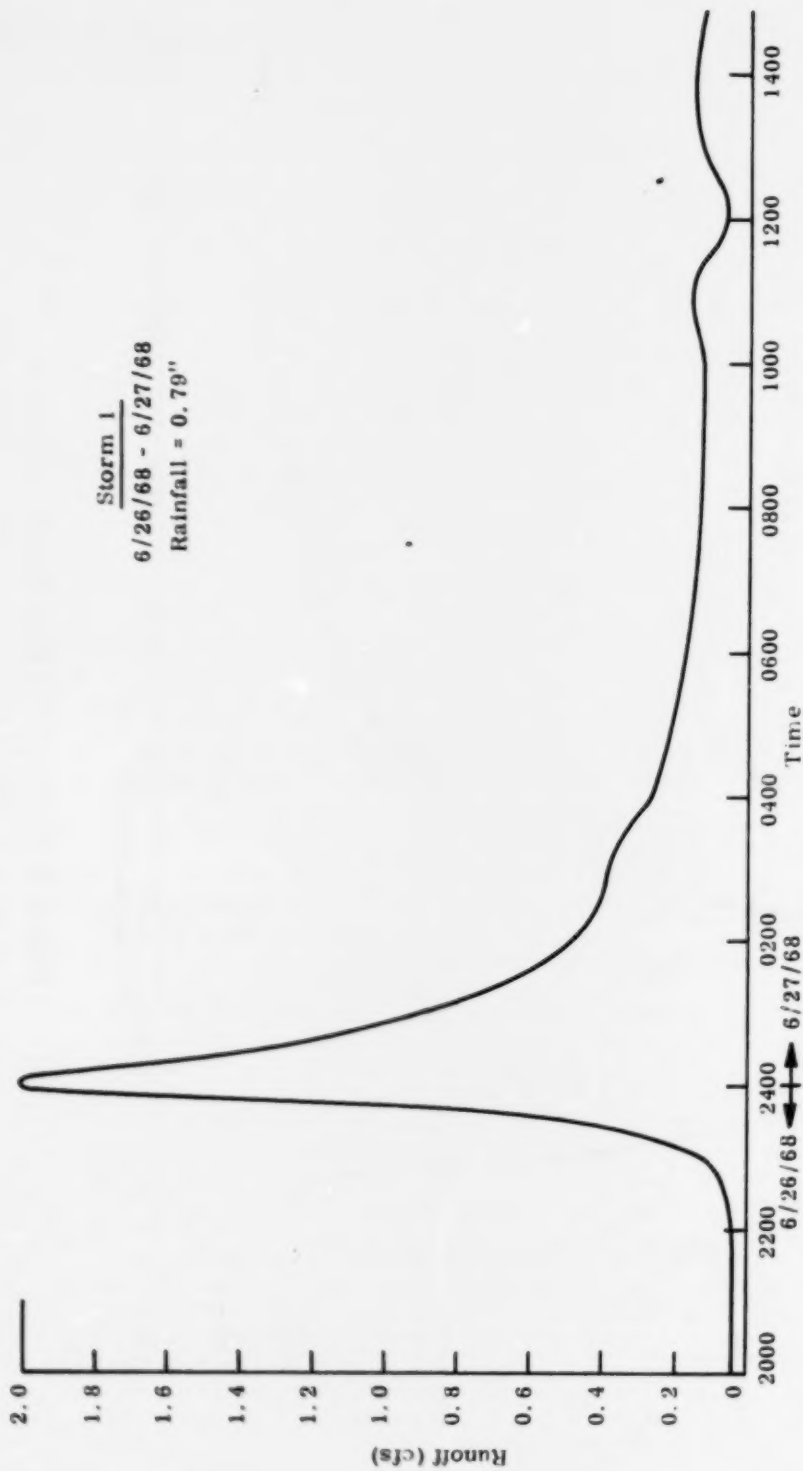


Figure B-3. Observed Hydrograph - Storm 1

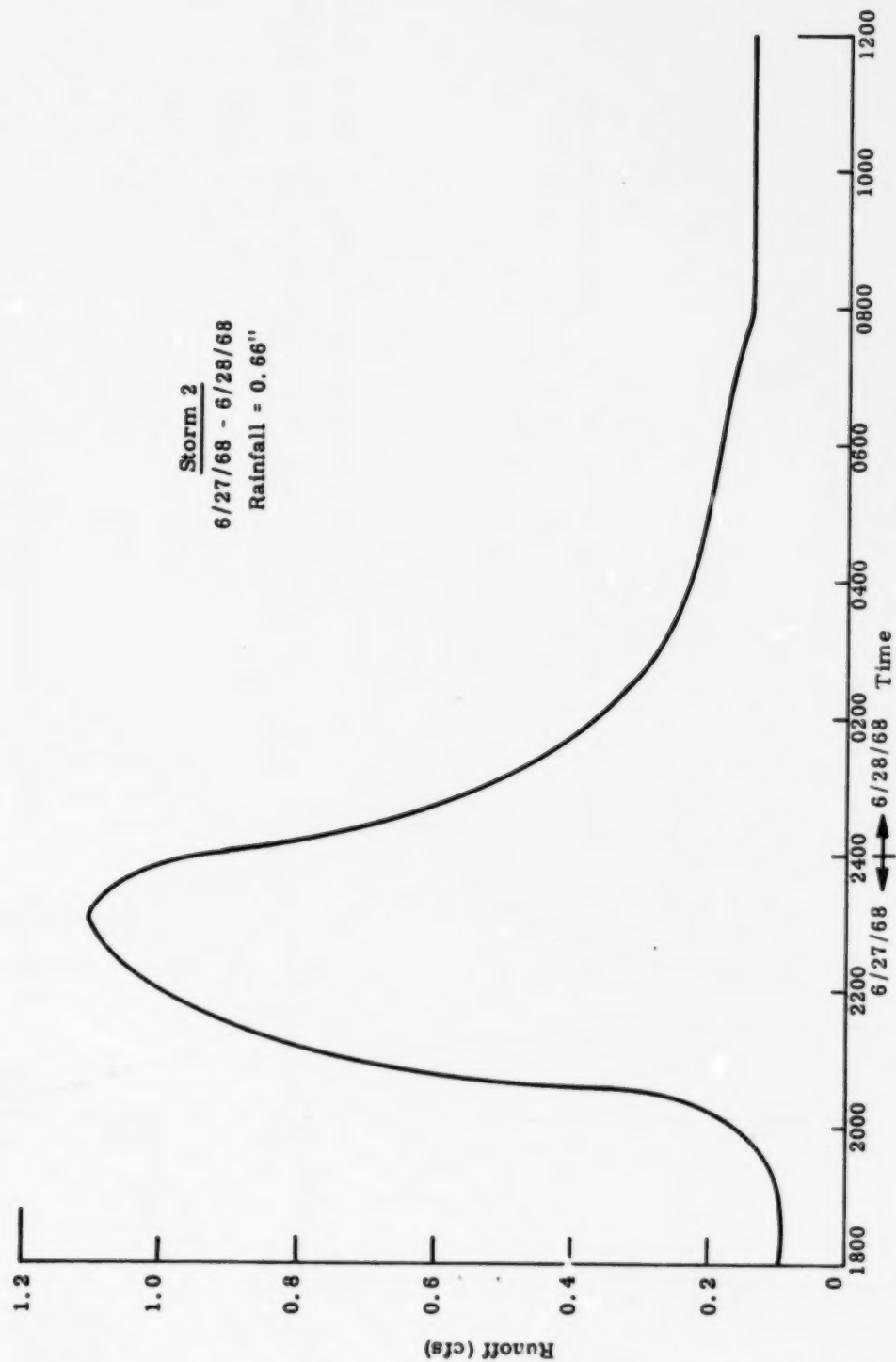


Figure B-4. Observed Hydrograph - Storm 2

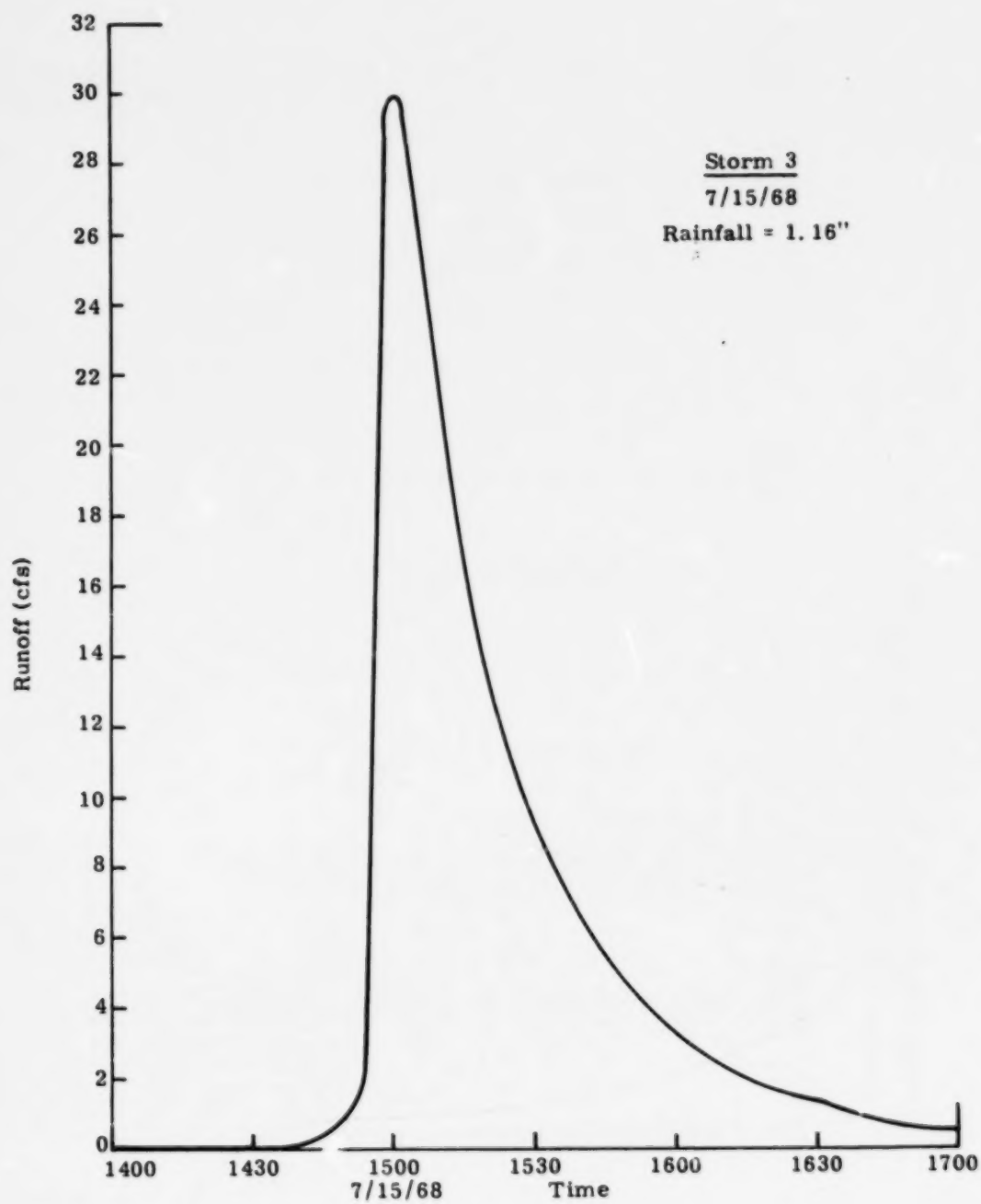


Figure B-5. Observed Hydrograph - Storm 3

TABLE B-2. WATER QUALITY VERSUS RUNOFF RATE
JUNE 26 TO JUNE 27, 1968 (STORM 1)

Sampling Time	Runoff Rate cfs	Turbidity JTU	Suspended Solids mg/l	Settable Solids mg/l	Total Solids mg/l	Chemical Oxygen Demand mg/l	pH	Alkalinity (CaCO ₃) mg/l	Hardness (CaCO ₃) mg/l	Chloride (Cl ⁻) mg/l	Nitrate (Nitrite) mg/l	Phosphate (PO ₄ ⁻) mg/l
16:30	0.04	190	140	---	252	20	8.3	55	50	8	0.7	---
17:30	0.04	125	---	---	---	---	---	---	---	---	---	---
18:30	0.04	78	50	---	---	---	8.3	---	---	---	---	---
19:30	0.04	130	---	---	---	20	8.3	55	50	7.5	0.5	---
20:30	0.04	82	75	---	185	---	---	---	---	---	---	---
21:30	0.04	103	---	---	---	---	---	---	---	---	---	---
22:30	0.04	85	80	---	---	30	---	---	---	---	---	---
23:30	1.00	4,500	11,450	10,830	11,650	40	7.9	55	45	7.5	0.6	3.4
24:30	1.65	3,500	5,760	5,250	---	---	7.1	20	30	24	0.1	0.3
01:30	0.60	1,500	1,950	1,550	2,100	30	7.2	20	25	12	0.0	1.1
02:30	0.44	1,050	1,105	920	---	20	7.3	25	25	11.5	0.2	---
03:30	0.33	820	823	---	970	30	7.6	25	30	12	0.3	---
04:30	0.23	325	290	---	---	---	---	---	---	---	---	---
05:30	0.20	270	---	---	---	---	---	---	---	---	---	---
06:30	0.17	---	---	---	---	---	---	---	---	---	---	---
07:30	0.16	151	150	---	255	---	8.0	30	35	10	---	---
08:30	0.15	---	---	---	---	---	---	---	---	---	---	---
09:30	0.14	235	238	---	---	30	8.2	35	40	9.5	0.6	---
10:30	0.15	103	---	---	---	---	---	---	---	---	---	---
11:30	0.13	1,050	925	---	1,110	---	---	---	---	---	---	---
12:30	0.10	310	310	---	---	---	---	---	---	---	---	---
13:30	0.15	105	---	---	---	---	---	---	---	---	---	---
14:30	0.14	1,300	1,500	1,220	1,680	30	7.2	10	30	10	0.5	1.7

TABLE B-3. WATER QUALITY VERSUS RUNOFF RATE
JULY 12, 1968 (BASE FLOW)

Sampling Time	Runoff Rate cfs	Turbidity JTU	Suspended Solids mg/l	Settable Solids mg/l	Total Solids mg/l	Chemical Oxygen Demand mg/l	pH	Alkalinity (CaCO ₃) mg/l	Hardness (CaCO ₃) mg/l	Chloride (Cl ⁻) mg/l	Nitrate (Nitrite) mg/l	Phosphate (PO ₄ ⁻³) mg/l
02:00	0.04	32	15	----	---	--	7.55	--	--	---	---	---
04:00	0.04	35	12	----	130	15	7.55	45	30	7.5	0.32	0.10
06:00	0.04	35	10	----	---	--	7.55	--	--	---	---	---
08:00	0.04	32	10	----	129	20	7.55	40	30	7.5	0.30	0.10
10:00	0.04	38	14	----	---	--	7.60	--	--	---	---	---
12:00	0.04	45	21	----	140	15	7.55	40	33	7.5	0.29	0.08
14:00	0.04	38	16	----	---	--	7.55	--	--	---	---	---
16:00	0.04	40	18	----	138	10	7.55	40	30	7.5	0.27	0.12
18:00	0.04	35	11	----	---	--	7.60	--	--	---	---	---
20:00	0.04	38	8	----	122	15	7.55	43	35	7.5	0.25	0.10
22:00	0.04	30	12	----	---	--	7.60	--	--	---	---	---
24:00	0.04	35	16	----	130	10	7.60	40	30	7.5	0.27	0.09

TABLE B-4. WATER QUALITY VERSUS RUNOFF RATE
JULY 15, 1968 (STORM 3)

Sampling Time	Runoff Rate Q cfs	Turbidity JTU	Suspended Solids mg/l	Settable Solids mg/l	Total Solids mg/l	Chemical Oxygen Demand mg/l	pH	Alkalinity (CaCO ₃) mg/l	Hardness (CaCO ₃) mg/l	Chloride (Cl ⁻) mg/l	Nitrate (Nitrite) mg/l	Phosphate (PO ₄ ⁼) mg/l
13:00	0.05	30	10	-----	128	10	7.50	40	30	7.5	0.28	0.10
14:30	0.05	35	20	-----	-----	10	6.95	43	30	7.5	0.31	0.30
15:10	19.50	5,200	23,800	23,300	23,950	10	6.55	15	30	15.0	0.15	0.07
15:20	12.50	4,600	14,600	13,600	-----	30	6.35	15	25	12.5	0.34	1.90
15:30	8.20	4,300	10,500	9,800	10,680	30	6.25	10	20	22.5	0.60	4.60
15:45	4.70	4,000	7,750	7,100	-----	20	6.50	8	25	10	0.20	0.50
16:00	2.70	3,600	6,000	5,380	6,200	40	6.80	10	25	22.5	0.60	5.30
16:15	2.20	2,600	3,900	3,500	-----	20	7.00	11	20	12.5	0.35	2.20
16:30	1.00	2,300	3,040	-----	3,170	30	7.10	10	25	10	0.34	0.15
17:30	0.30	1,100	955	-----	-----	40	7.00	10	23	9	0.45	0.18
18:30	0.30	1,150	1,030	-----	-----	20	7.05	18	25	9	0.27	0.27
19:30	0.20	750	725	-----	-----	10	7.15	20	28	9	0.28	0.20
20:30	0.20	1,900	1,510	-----	1,660	10	7.25	25	35	14	0.32	0.15
14:30	0.08	58	50	-----	-----	5	7.45	35	30	7.5	0.28	0.25

facilities were not available for determining BOD, and a COD test was employed instead. Due to the atypical conditions existing, however, the results of the tests during storm flows are judged to have little value with respect to estimating future pollution loads. The data presented on Table B-3 is of some interest in that the base flow water quality might be expected to be relatively independent of watershed conditions. As the data indicate, this flow was found to be of excellent quality, well within the expected limits. The continuation of this sampling activity throughout the period of urbanization of this watershed would be of considerable value in assessing the effect of construction and subsequent population on runoff quality.

APPENDIX C

STORM WATER STORAGE AND PRETREATMENT COSTS

RESERVOIR CONSTRUCTION COSTS

Construction costs were calculated for the five types of storm water storage reservoirs listed in Table 12. All types except the open ponds have an alternate application as treated water storage facilities and the same construction costs were employed for both applications. In addition, other types of storage were considered for treated water only, namely, standpipes, elevated tanks, and hydropneumatic tanks. Table C-1 lists all types of storage considered for either application. This section will describe the methods employed to compute parametric costs for each of these storage types. The following section will outline operation and maintenance costs for the storm water application only. Similar costs for treated water storage applications can be found in Appendix D.

TABLE C-1. RESERVOIR CONSTRUCTION TYPES

<u>Construction Type</u>	<u>Application</u>	
	<u>Storm Water</u>	<u>Treated Water</u>
Natural Storage		
Open Pond	X	
Ground Level Storage		
Steel Tank - Above Grade	X	X
Steel Tank - Below Grade	X	X
Concrete Tank - Above Grade	X	X
Concrete Tank - Below Grade	X	X
Elevated Storage		
Standpipe		X
Elevated Tank		X
Pressure Storage		
Hydropneumatic Tank		X

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Construction costs for storage facilities were considered to be made up of the following:

1. Land costs, applicable wherever the purchase of the land is a necessary cost of the system.
2. Site preparation costs, including clearing and grubbing, rough grading, finish grading, and seeding or sodding after construction.
3. Burial costs, including excavation and backfill for tanks which are installed below grade.
4. Erection costs, which include foundations, tank erection, painting of metal parts, and in the case of ponds, necessary excavation, fill, embankment, spillway construction, and bottom and slope preparation.

In each case, actual estimates were made of construction costs for a number of hypothetical installations of various types and sizes in the Columbia area. Prices of 1967 to 1968 were used and allowances made for engineering, inspection, and construction contingencies. The estimates were then plotted against the appropriate size parameter and a smooth curve extended between the points. This curve was then replaced by a series of linear approximations which closely followed it and the inflection points reduced to a table of sizes and costs. This table was input to the computer in the systems analysis, so that the cost of any size facility could be estimated by a linear interpolation between two points on the table. In general, the original estimates were made at sizes which permitted the curve to be well defined in the range of possible application in the systems analysis.

LAND COSTS

In most applications, land can be purchased for a per-acre price which is relatively uniform throughout the project area. Methods for computing the required land area for each type and size of tank or pond were developed, and costs were determined by simply permitting the computer program to multiply the land area in acres by the cost in dollars per acre, which was input separately. Computation of required land area for open ponds required the conceptual siting of a variety of pond sizes in the Wilde Lake Watershed and the computation of the land area. In every case, a 30-foot border was provided beyond the high water line to protect slopes and permit decorative fringing with trees and shrubbery. No attempt was made to force areas into regular shapes except for ease of computation. Figure C-1 shows the resulting land area/capacity curve, with the linear approximation overlaid. Table C-2 lists the inflection points of the linear approximation which were input to the computer for pond land area computations. All other types of storage were treated as cylindrical tanks and their land requirements were computed on the

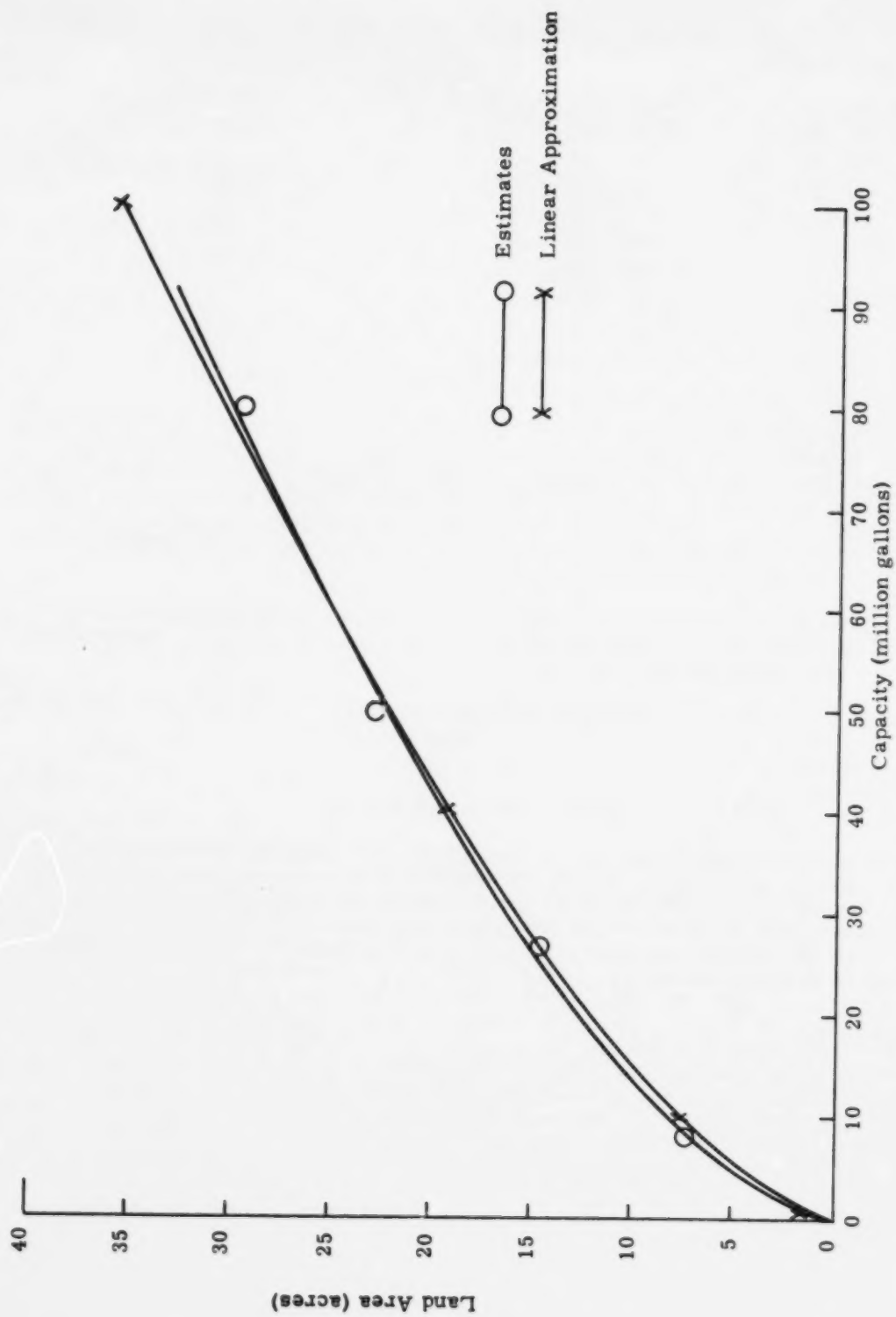


Figure C-1. Land Area vs. Capacity - Open Ponds

TABLE C-2. LAND AREA VS. CAPACITY - OPEN PONDS

<u>Capacity (gallons)</u>	<u>Land Area (acres)</u>
0.0	0.0714
10,000	0.0714
100,000	0.340
1,000,000	1.690
4,000,000	4.110
10,000,000	7.645
20,000,000	12.250
40,000,000	19.550
100,000,000	36.370

basis of height/diameter (H/D) ratios. These ratios, found by dividing tank height by tank diameter, were determined by plotting H/D for standard designs of a variety of tanks. Figure C-2 shows such a plot for ground level tanks and elevated tanks. Figure C-3 shows H/Ds for standpipes. The corresponding linear approximations can also be seen on these figures, and Tables C-3, C-4, and C-5 list the ratios used in the computer program.

Required land area is found by adding 30 feet to the diameter, thus providing for a minimum 15-foot border around the tank. This calculation is performed as follows:

$$\text{BAR} = 0.000018 (\text{DIAM} + 30.0)^2 \quad (\text{C-1})$$

where:

$$\text{BAR} = \text{required land area, acres}$$

Since the land to be used in the Wilde Lake Watershed was entirely public open space, the initial systems analysis computer runs were made with land cost set at zero. Should the value of land to be used for system construction become a significant consideration, insertion of the appropriate per-acre cost into the computer input will result in revised estimates.

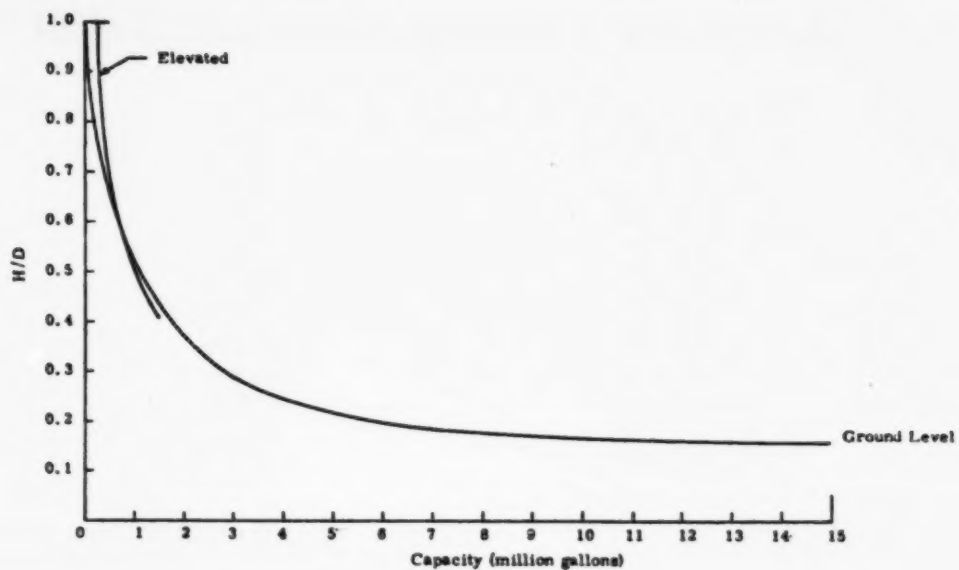


Figure C-2. H/D vs. Capacity - Ground Level and Elevated Tanks

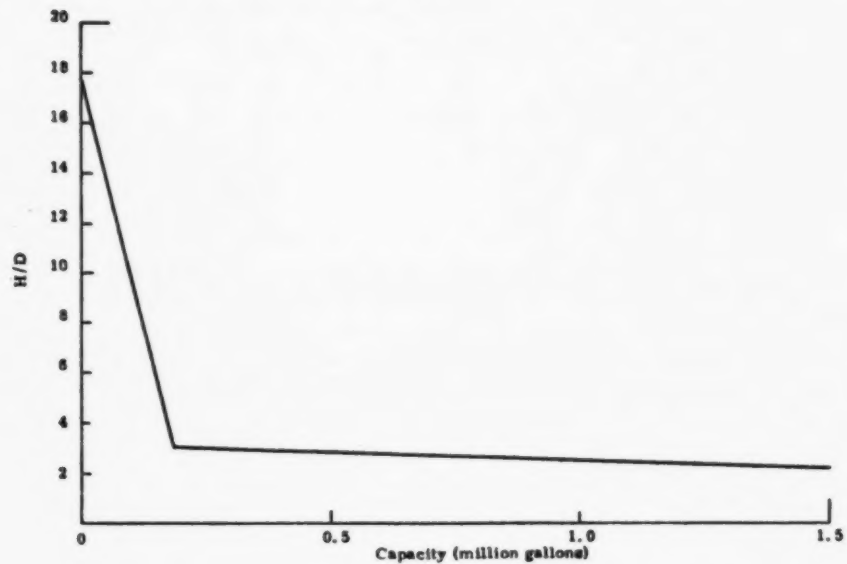


Figure C-3. H/D vs. Capacity - Standpipes

TABLE C-3. H/D VS. CAPACITY - GROUND LEVEL TANKS

<u>Storage Capacity (gallons)</u>	<u>H/D</u>
0.0	1.000
50,000	0.900
200,000	0.759
500,000	0.640
1,000,000	0.512
2,000,000	0.366
3,000,000	0.285
5,000,000	0.210
7,000,000	0.180
10,000,000	0.165
100,000,000	0.0348

TABLE C-4. H/D VS. CAPACITY - ELEVATED TANKS

<u>Storage Capacity (gallons)</u>	<u>H/D</u>
0.0	0.995
250,000	0.995
300,000	0.900
500,000	0.710
700,000	0.588
1,000,000	0.490
1,250,000	0.442
1,500,000	0.410

TABLE C-5. H/D VS. CAPACITY - STANDPIPES

<u>Capacity (gallons)</u>	<u>H/D</u>
0.0	17.25
180,000	3.00
1,500,000	2.20

SITE PREPARATION COSTS

Since the costs treated under this heading are primarily related to land area and not to type of construction, they were evaluated on a per-acre basis and applied to all tanks. The single exception to this is the case

of open ponds, where site preparation costs were considered as part of erection costs and included in that tabulation. Figure C-4 shows the curve plotted from estimates and the linear approximation. The table of values used in the computer program is shown as Table C-6. Land area is computed as described in the previous paragraph on land costs. Per-acre site preparation costs corresponding to various capacities are obtained by linear interpolation of the table. The interpolated value is then multiplied by the land area to yield total site preparation cost.

TABLE C-6. SITE PREPARATION COSTS VS. CAPACITY

<u>Capacity (gallons)</u>	<u>Site Preparation Costs (\$/acre)</u>
0.0	34,500
100,000	27,750
300,000	20,300
700,000	14,200
1,150,000	11,000
1,500,000	9,500
2,000,000	8,200
2,500,000	7,400
10,000,000	3,400
100,000,000	3,400

BURIAL COSTS

Burial costs are computed only for the ground level storage tanks installed below grade. In these cases, the total cost of excavation and backfill associated with the construction of various underground tanks was estimated and the results plotted as Figure C-5. This figure also shows the linear approximation of the curve which is represented by Table C-7. The tank capacity is used to interpolate between points on the table, yielding burial costs in dollars per gallon. When this figure is multiplied by the capacity, the total burial costs are obtained.

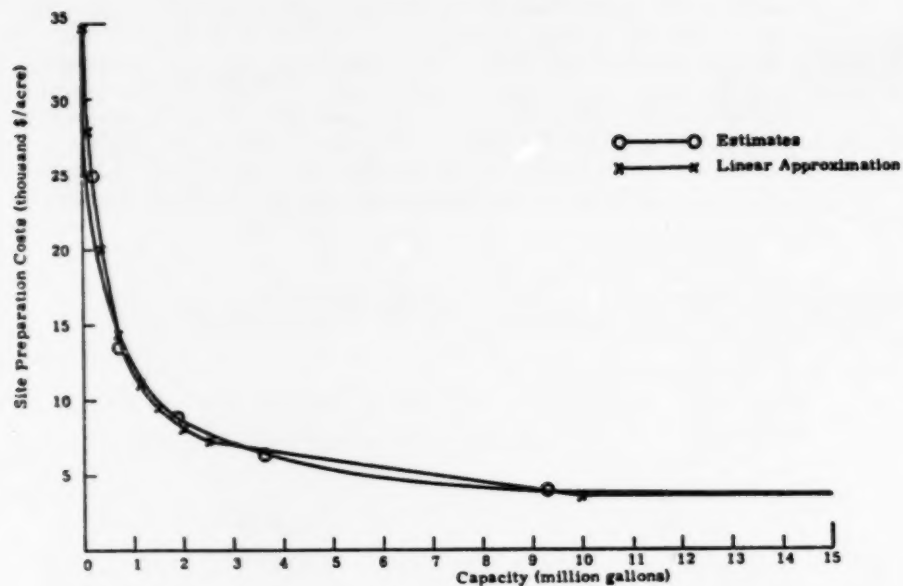


Figure C-4. Site Preparation Costs vs. Capacity - Storage Tanks

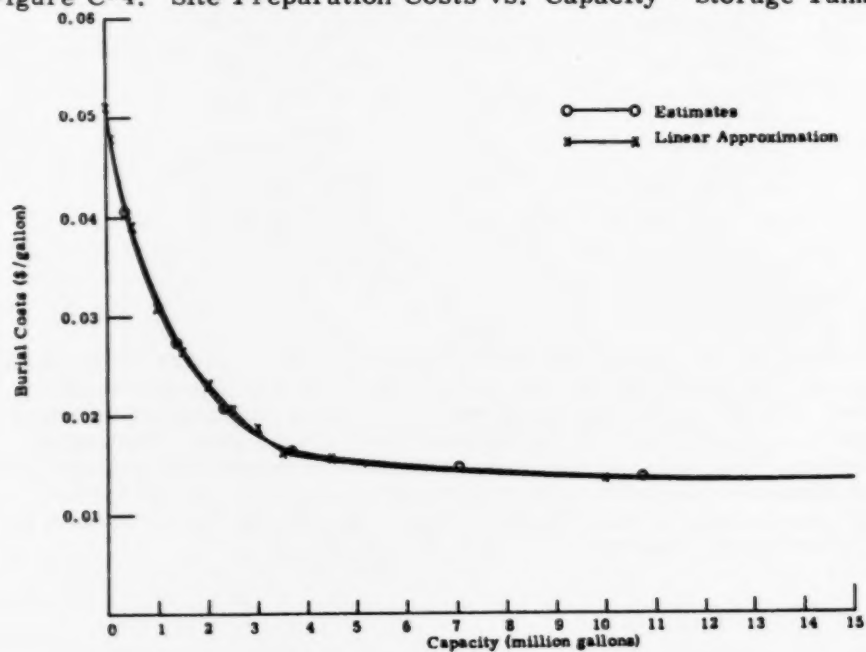


Figure C-5. Burial Costs vs. Capacity - Storage Tanks

TABLE C-7. BURIAL COSTS VS. CAPACITY

<u>Capacity (gallons)</u>	<u>Burial Costs (\$/gallon)</u>
0	0.0510
100,000	0.0480
500,000	0.0390
1,000,000	0.0310
1,500,000	0.0269
2,000,000	0.0230
2,500,000	0.0206
3,000,000	0.0188
3,500,000	0.0176
4,500,000	0.0162
10,000,000	0.0134
100,000,000	0.0134

ERECTION COSTS

Estimates were obtained from tank fabricators and erectors for in-place costs of a variety of tank types and sizes. Table C-8 is a summary of that information. Figure C-6 shows curves for various types of above grade and elevated tanks. Tanks designed for burial must be constructed somewhat differently to account for the additional stresses, and information from fabricators indicated an approximate 15 percent increase in the tank cost. When the burial option is selected in the program, then 15 percent is added to the tank erection cost in addition to the computation of the burial cost. Table C-9 lists the program inputs used to describe the erection cost capacity relationships for various tank types.

In the case of open ponds, the erection cost is highly site-dependent. Furthermore, a tradeoff exists between excavation of the site to increase storage and providing additional spillway elevation. In addition to being dependent upon the relative costs of excavation and spillway and embankment construction, the optimum combination is a function of the site topography and the spillway-embankment width. In order to arrive at an approximate parametric relationship between pond capacity and cost, a site was selected from the Wilde Lake watershed (sub-watersheds number 4 and 5) and a dam location determined. Using a large-scale topographic map with one-foot contours, pond capacity was determined for each foot of elevation of the dam up to the maximum feasible elevation at the site. These capacities were those of completely natural ponds with

TABLE C-8. SUMMARY OF TANK ERECTION COST ESTIMATES

	Capacity (gallons)	Erection Costs (\$)
<u>Ground Level Tanks - Steel</u>	50,000	10,000(1) 8,400(2)
	100,000	13,000(1) 11,000(2)
	200,000	15,500(2)
	500,000	30,000(1) 27,000(2)
	1,000,000	53,000(1) 44,000(2)
	2,000,000	110,000(1) 77,000(2)
	5,000,000	175,000(2)
	10,000,000	340,000(2)
<u>Ground Level Tanks-Concrete</u>	100,000	18,500(3)
	250,000	28,000(3) 45,000(4)
	500,000	45,000(3) 68,000(4)
	750,000	80,000(4)
	1,000,000	69,000(3) 100,000(4)
	2,000,000	125,000(3) 150,000(4)
	5,000,000	250,000(3) 325,000(4)
	10,000,000	500,000(3) 500,000(4)
<u>Elevated Tanks</u>	50,000	38,000(1) 30,000 ↔ 33,000(2)
	100,000	45,000(1) 38,500 ↔ 44,000(2)
	200,000	60,000(1) 42,500 ↔ 52,500(2)
	500,000	110,000(1) 85,000 ↔ 95,000(2)
	1,000,000	220,000(1) 170,000 ↔ 186,000(2)
	2,000,000	375,000(1) 290,000 ↔ 330,000(2)
<u>Standpipes</u>	100,000	19,000(1)
	500,000	45,000(1)
	1,000,000	80,000(1)
	2,000,000	130,000(1)

- (1) Pittsburgh-Des Moines Steel Co.
- (2) Chicago Bridge and Iron
- (3) The Crom Co.
- (4) The Preload Co.

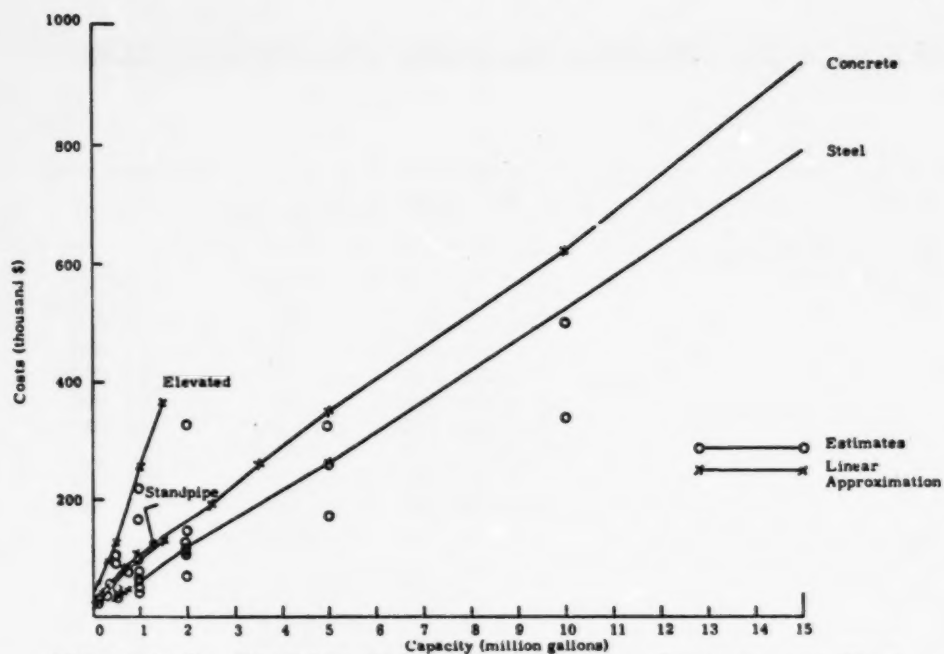


Figure C-6. Erection Costs vs. Capacity - Concrete and Steel Above Ground Tanks, Elevated Tanks, and Standpipes

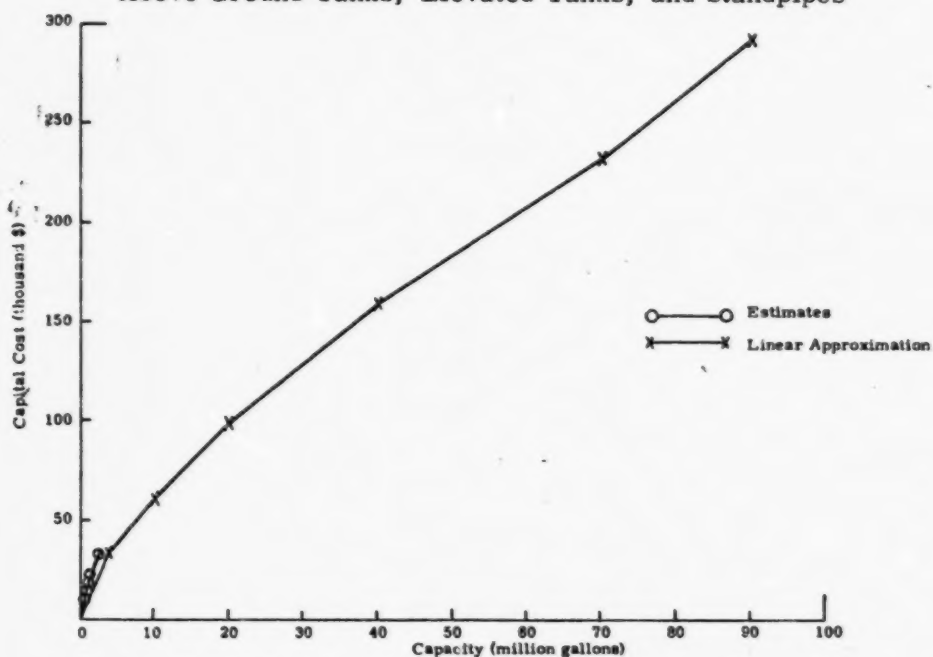


Figure C-7. Construction Costs vs. Capacity - Open Ponds

TABLE C-9. ERECTION COSTS VS. CAPACITY - STORAGE TANKS

	Capacity (gallons)	Erection Cost (\$)
<u>Ground Level Storage</u>		
<u>Steel</u>	0.0	5,000
	10,000	5,000
	50,000	11,000
	200,000	21,000
	400,000	32,000
	700,000	48,000
	1,000,000	65,000
	2,000,000	122,000
	5,000,000	263,000
	100,000,000	5,260,000
<u>Ground Level Storage</u>		
<u>Concrete</u>	0.0	15,000
	10,000	15,000
	50,000	20,000
	100,000	25,000
	300,000	51,000
	900,000	100,000
	1,500,000	139,000
	2,500,000	197,000
	3,500,000	259,000
	5,000,000	355,000
	10,000,000	625,000
	100,000,000	6,250,000
<u>Elevated Tanks</u>	0.0	26,000
	10,000	26,000
	50,000	45,000
	100,000	55,000
	200,000	72,000
	350,000	102,000
	500,000	132,000
	1,000,000	258,000
	1,500,000	367,000
<u>Standpipes</u>	0.0	15,000
	10,000	15,000
	50,000	20,000
	200,000	32,000
	400,000	51,000
	700,000	82,000
	1,500,000	137,000

no excavation. Next, capacities were recalculated for each elevation with excavation of the basin until the incremental cost of excavation (in dollars per gallon) equaled the incremental cost of the next change in dam elevation. These final capacities were used to develop the cost versus capacity curve which appears in Figure C-7. In general, excavation accounted for about one-third of the final pond volume. A summary of the cost estimates appears in Table C-10. Table C-11 has the final values used in the computer input.

OPERATION AND MAINTENANCE

In the preparation of the preliminary parametric cost data for use in the computer program, the operation and maintenance costs for the storm water storage facilities were considered to be under three headings. These were:

1. Routine checks of storage and pretreatment facilities
2. Trash removal from storage facilities
3. Sediment removal from storage and pretreatment facilities

In this initial analysis, it was assumed that the routine checks of the facilities would be conducted approximately once per week and would require one-half man-hour per reservoir. It was assumed that trash removal would be required on half of the inspections and an additional two-tenths of a man-hour were allotted for these inspections.

In the generation of the parametric cost data, it was assumed that sediment would be removed by pumping from the tanks and pretreatment facilities using tank trucks. One hour was allotted for this operation requiring a semiskilled laborer and an equipment operator. It was assumed that this operation would be required 25 times per year and that the tank truck rental would be \$100 per day or \$8.33 per hour or per operation.

For the disposal of the sediment, it was assumed that a satisfactory dumping site would be available within two miles of the reservoir. No allowance was made for the operation and maintenance of a sediment disposal facility in this study.

The labor rates used in this initial analysis were based on the use of semiskilled laborers at the rate of \$2.50 per hour, plus 15 percent for fringe benefits, or \$2.88 per hour, for the routine checks, trash removal, and as a helper on the sediment removal operations. The labor rate for the equipment operator was estimated at \$3.50 per hour, plus 15 percent, or \$4.03 per hour.

Table C-12 summarizes the estimated cost of the routine checks and trash and sediment removal used in the development of the parametric

TABLE C-10. SUMMARY OF CONSTRUCTION COST ESTIMATES - OPEN PONDS
(Cost Items in Dollars)

Capacity (gal.)	Concrete Yards	Excavation Yards	Round Area (sq. ft.)	Concrete Cost	Embank- ment Cost	Excavation Cost	Clear & Grub	Rip Rap	Net Total	Cont. +25%	Gross Total
52,950	16.02	87.9	2,500	1,153	30	160	30	109	1,482	371	1,853
97,950	20.11	161.9	4,000	1,448	30	360	40	402	2,280	570	2,850
159,450	23.89	263.8	5,500	1,720	30	535	55	740	3,080	775	3,855
238,800	39.22	393.4	7,000	2,820	54	810	70	786	4,540	1,135	5,675
339,300	42.77	560.1	9,000	3,078	60	1,050	90	1,142	5,420	1,355	6,775
465,000	52.03	768.4	11,250	3,746	71	1,320	112	1,031	6,280	1,570	7,850
630,000	70.37	1,037.0	15,000	5,063	101	1,300	150	970	7,584	1,896	9,480
858,000	87.00	1,416.7	20,500	6,226	497	608	207	1,878	9,454	2,364	11,818
1,015,000	102.00	1,660.0	23,000	7,344	520	1,850	230	1,536	11,480	2,870	14,305
1,105,000	110.00	1,800.0	24,400	7,920	529	2,040	244	1,187	11,920	2,980	14,900
1,780,000	151.00	2,850.0	35,000	10,872	840	3,358	350	3,500	18,920	4,730	23,650

TABLE C-11. CONSTRUCTION COST VS. CAPACITY - OPEN PONDS

<u>Capacity (gallons)</u>	<u>Construction Costs (\$)</u>
0.0	1,765
50,000	1,765
200,000	5,000
1,000,000	13,290
4,000,000	33,800
10,000,000	62,600
20,000,000	99,800

TABLE C-12. SUMMARY OF SEDIMENT REMOVAL COSTS

	<u>Number of Occurrences per Year</u>	<u>Man- hrs</u>	<u>Rate</u>	<u>Labor Cost</u>	<u>Equip. Cost</u>	<u>Per Occurrence</u>	<u>Total per Year</u>
Routine Check	50	0.5	2.88	1.44	0.50	1.94	\$ 97.00
Trash Removal	25	0.2	2.88	0.58	----	0.58	14.50
Sediment Removal	25	1.0 1.0	2.88 4.03	2.88 4.03	---- 8.33	---- 11.47	<u>381.00</u>
Total trash and sediment removal, cost per year							\$492.50
Trash and sediment removal, cost per gallon runoff							\$0.00004925

cost data for the computer program. These costs are based on a storage facility receiving 10,000,000 gallons of runoff per year.

It is noted that this study was based on reservoirs operating in an established and stabilized community. As such, the estimates were not based on the quantities of sediment that would be generated during construction. This is further discussed in Section VII.

The parametric cost data selected and used in the computer program were \$0.00005 per gallon for runoff for both the tank type storage and open ponds, independent of size. As discussed in Section XII, the operating and maintenance costs for the storage reservoirs and pretreatment unit were further refined in making the cost estimates for the various local storage and reuse systems. In these estimates, the costs for sediment removal from the storage reservoirs and pretreatment unit were separately estimated and the frequency of the various operations and man-hour estimates were revised based on the actual design. These later operation and maintenance costs also include the cost of chemicals used in the pretreatment units and the labor and materials for preventive maintenance and repair.

In addition to the costs of the routine checks and cleaning of the storage reservoir and pretreatment unit, the parametric cost input included provision for painting and repair of metal surfaces, hatchways, doors, etc., as well as repair to concrete structures. Ground maintenance was not included since the normal maintenance that would be given to the open space where the facilities are located is expected to be adequate. Minor repairs and painting can be accomplished as part of the routine inspections noted above. Concrete structures are not expected to require significant maintenance during the time span of the economic analysis. Metal tanks, however, must be periodically painted and repaired. Buried tanks require maintenance of their internal surface, while tanks installed above grade have two surfaces to be maintained. Repair and maintenance experiences with a number of tanks in the Baltimore area were supplied by Whitman, Requardt and Associates, along with the number of square feet maintained in each case. The total cost, taking into account painting and repairing exposed metal surfaces, has been found to average \$0.05 per square foot per year.

The inside surface area of buried tanks was computed as follows:

$$\text{SURI} = 3.14159 \text{ DIAM (HIGH} + 0.5 \text{ DIAM)} \quad (\text{C-2})$$

where:

HIGH = tank height = tank diameter times H/D ratio, feet

SURI = total inside surface area, square feet

This area includes inside walls, floor, and ceiling. The outside area of above-grade tanks was computed to include outside walls and roof as follows:

$$\text{SURO} = 3.14159 \text{ DIAM (HIGH} + 0.25 \text{ DIAM)} \quad (\text{C-3})$$

where:

SURO = total outside surface area, square feet

Total surface area of tanks above grade was taken as inside area plus outside area, or SURI + SURO. Maintenance costs are then computed by multiplying tank area by \$0.05 per square foot per year.

APPENDIX D

TREATMENT SYSTEM COSTS

EQUIPMENT AND FACILITY COSTS

Construction costs for pretreatment, final treatment, pumping, and treated water storage were determined by computing estimated costs for each of a series of design capacities and assuming that a continuous function exists between the calculated points. In each case, a minimum capacity was determined, below which it was assumed that costs would not decrease with decreasing capacity. Underground construction within the floodplain area was assumed for pretreatment and final treatment units, both to minimize pumping costs and overflow provisions and to permit unobtrusive installations in residential neighborhoods. Underground, ground level, and elevated options were considered for treated water storage.

PRETREATMENT

Estimated construction costs for the pretreatment unit are listed in Table D-1. The pH adjustment process was not included in the parametric costs, due to its nominal cost and the high probability of a satisfactory pH at the Columbia location. Costs are based on a solution feed of both coagulant and chlorine, obtained from hypochlorite. Use of chlorine gas was also investigated, but not included in the parametric cost analysis due to the wide range of capacities being investigated and the probable inapplicability of chlorine gas feed at the smaller size.

The complete pretreatment construction cost/capacity function was approximated by fitting a curve to the points developed in Table D-1. This curve was, in turn, approximated by a series of inflection points and straight lines, permitting the data to be input to a computer program as a table of cost/capacity relationships. This table is shown as Table D-2 and the two curves are illustrated on Figure D-1.

FINAL TREATMENT

Capital costs for the final treatment phase were developed in the same manner as pretreatment costs. Facilities for each treatment level were estimated for several capacities within the range of those being investigated. A capital cost/capacity curve was approximated and this curve, in turn, approximated by a series of straight lines. The inflection points of this last curve were reduced to a table for input to the computer program, permitting calculation of a cost for any size plant by a simple linear interpolation between the appropriate points on the table. The basic cost data are summarized on Table D-3, and Figure D-2 shows the developed curves for each of treatment levels. Since no final

TABLE D-1. SUMMARY OF CONSTRUCTION COST ESTIMATES - PRETREATMENT

Capacity (gal/day)	(Costs in Dollars)						Total Cost
	Excavation and Backfill	Structure, Pipes, Etc.	Tube Settler	Chemical Feeders	Construction Cost	Engineering, Contractor, Etc. (@25%)	
21,600	150	315	125	250	840	210	1,050
130,000	280	650	750	400	2,000	520	2,600
650,000	400	3,350	3,750	500	8,000	2,000	10,000
1,300,000	750	5,450	7,500	500	14,200	3,550	17,750

TABLE D-2. CONSTRUCTION COST VERSUS CAPACITY - PRETREATMENT

Pretreatment Capacity (overflow rate-gallons/day)	Capital Cost (dollars)
15,000	1,050
60,000	1,680
150,000	2,800
600,000	9,400
1,500,000	22,200
5,500,000	42,400

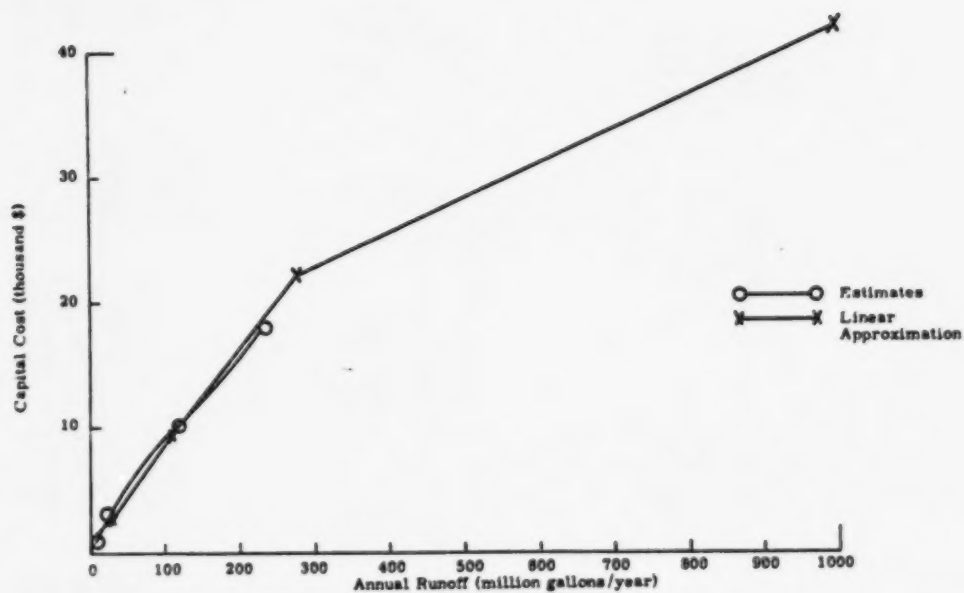


Figure D-1. Pretreatment Capacity vs. Capital Costs

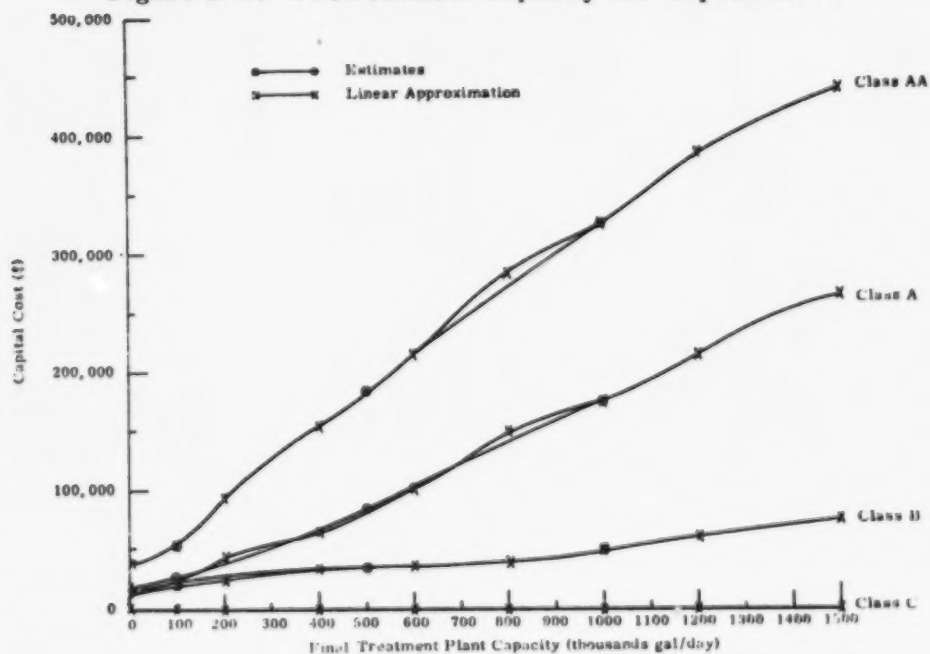


Figure D-2. Final Treatment Capacity vs. Capital Cost

TABLE D-3. SUMMARY OF CONSTRUCTION COST ESTIMATES - FINAL TREATMENT
(Cost in Dollars)

TREATMENT
CLASS "B"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Chemical Feeders	Strainer	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100,000	500	3,500	400	12,400	16,800	4,200	21,000
500,000	1,000	4,800	500	22,500	28,800	7,200	36,000
1,000,000	1,400	6,000	650	31,950	40,000	10,000	50,000

CLASS "A"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Chemical Feeders	Filter	Monitors	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100,000	5,000	9,800	400	3,800	1,000	20,000	5,000	25,000
500,000	14,000	38,000	500	12,000	1,000	66,000	16,500	82,000
1,000,000	22,000	89,350	650	27,000	1,000	140,000	35,000	175,000

CLASS "AA"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Filter	Chemical Feeders	Auto. Monitors	Activated Carbon Unit	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100,000	8,000	15,500	3,800	400	5,300	11,000	44,000	11,000	55,000
500,000	23,000	61,000	12,000	500	6,500	45,000	148,000	37,000	185,000
1,000,000	35,500	125,000	27,000	650	6,500	65,000	260,000	65,000	325,000

treatment plant is required for Class "C" water, costs are shown for Classes "B," "A," and "AA" only. Table D-4 shows the final costs input to the computer program.

TABLE D-4. CONSTRUCTION COST VERSUS CAPACITY -
FINAL TREATMENT

	Final Treatment Capacity (gal/day)	Capital Cost (dollars)
CLASS "B"	0.0	18,000
	100,000	21,000
	200,000	25,000
	400,000	34,000
	600,000	37,500
	800,000	40,000
	1,000,000	50,000
	1,200,000	60,000
	1,500,000	75,000
CLASS "A"	0.0	15,000
	100,000	25,000
	200,000	45,000
	400,000	65,000
	600,000	100,000
	800,000	150,000
	1,000,000	175,000
	1,200,000	215,000
	1,500,000	265,000
CLASS "AA"	0.0	40,000
	100,000	55,000
	200,000	95,000
	400,000	155,000
	600,000	215,000
	800,000	285,000
	1,000,000	325,000
	1,200,000	385,000
	1,500,000	440,000

As in the case of the pretreatment unit, underground construction and solution feed of all chemicals are assumed. Softening and secondary coagulation are not included in these costs. Chlorine costs are not shown separately, based on the assumption that the facility installed in the pretreatment unit would provide chlorine to both injection points.

Both Classes "A" and "AA" treatment facilities will include continuous effluent turbidity monitors to insure proper operation of the filters. This monitor will operate recorders, producing a permanent record of plant operation, as well as alarm systems which will be arranged to shut down the plant in the event of malfunction. It is anticipated that the maximum effluent turbidity can be maintained under 1.0 Jackson Turbidity Unit by the use of this control system.

Costs for Class "AA" treatment include more generous allowances for equipment and instrumentation. It is anticipated that product water from this type of installation would be introduced into an existing potable water distribution system, thus imposing a higher standard of reliability and fail-safe operation.

PUMPING

Costs were developed for the high-lift pumping requirement only. The below-grade installation of the treatment units permits gravity feed between processes and the cost of backwash or flushing pumps has been included in the cost of the related piece of equipment. Pump costs include control and switchgear costs and are based on a single pump for each application. Redundant facilities were not considered due to the supplemental and decentralized nature of the facilities under study. Similarly, it was not considered appropriate to cost premium-quality, continuous-duty equipment. Table D-5 summarizes the pump cost estimates and Figure D-3 shows the development of the final cost table. This final table is shown as Table D-6.

TABLE D-5. SUMMARY OF CONSTRUCTION COST ESTIMATES - PUMPING FACILITIES (Cost in Dollars)

Capacity (gal/day)	Pump & Motor	Controls Switchgear & Wiring	Valves & Appurt.	Const. Cost	Eng., Cont., Etc. (@25%)	Total Cost
1,000	110	30	20	160	40	200
28,800	315	50	35	400	100	500
72,000	570	300	250	1,120	280	1,400
144,000	1,220	400	300	1,920	480	2,400
720,000	4,940	700	600	6,240	1,560	7,800
1,440,000	10,000	1,400	1,000	12,400	3,100	15,500

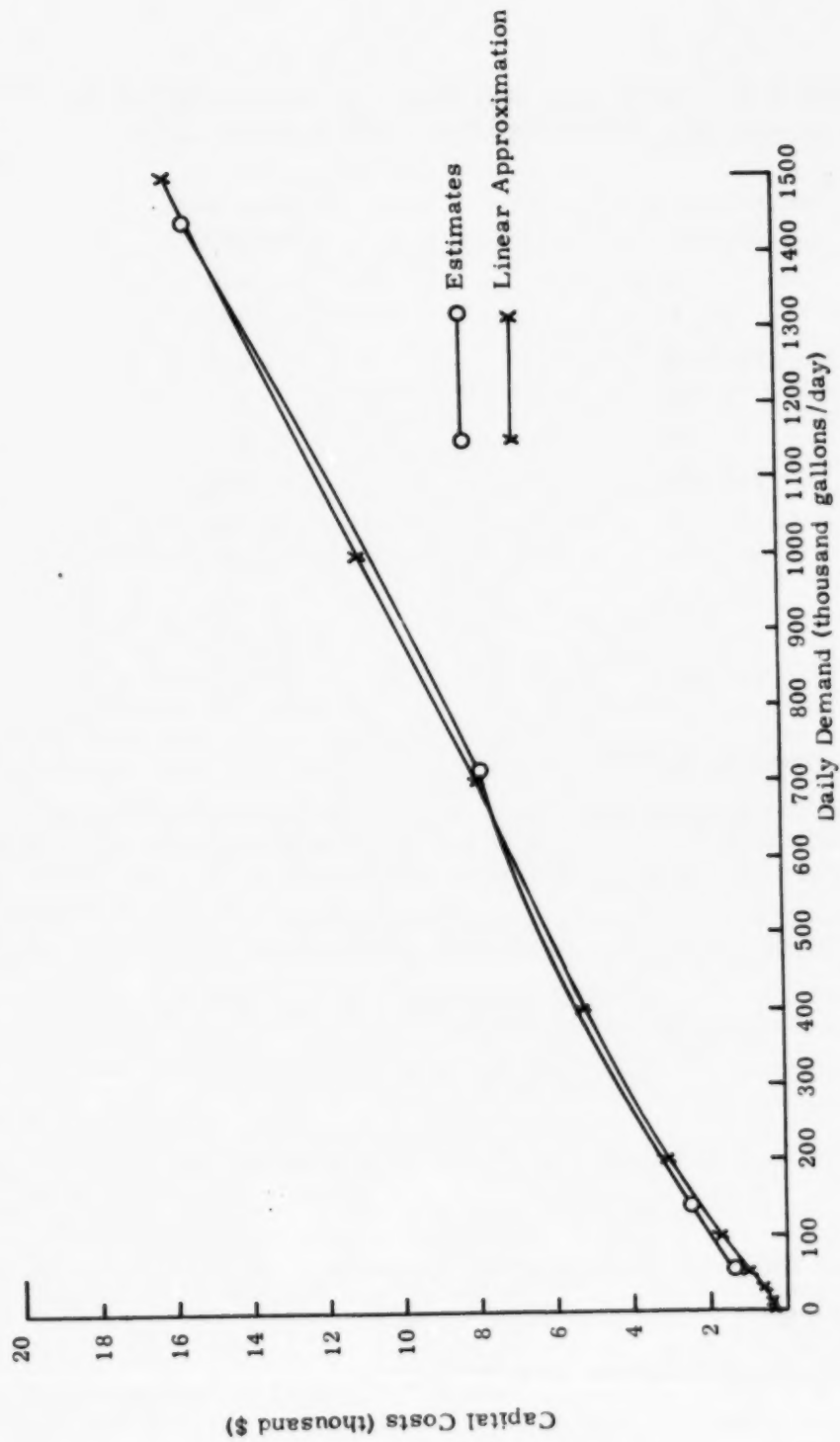


Figure D-3. Daily Demand vs. Capital Cost of Pumping Facilities

TABLE D-6. CONSTRUCTION COST VERSUS CAPACITY -
PUMPING FACILITIES

<u>Pumping Capacity (gal/day)</u>	<u>Capital Cost (dollars)</u>
0.0	200
1,000	200
10,000	250
30,000	600
50,000	1,000
100,000	1,840
200,000	3,100
400,000	5,300
700,000	8,000
1,000,000	11,000
1,500,000	16,000

TREATED WATER STORAGE

Seven different configurations of treated water storage were considered and costs developed for each of them. These alternatives can be divided into three different modes of operation, namely:

1. Ground level storage with direct pumping into distribution system, using storage as suction well
2. Elevated storage supplied by pumping, storage floating on the distribution system
3. Hydropneumatic tank floating on system and supplied by pumping

The seven storage configurations related to these modes are:

1. Ground level - steel tank below grade
2. ground level - concrete tank below grade
3. Ground level - steel tank above grade
4. Ground level - concrete tank above grade
5. Elevated - elevated tank on legs or pedestal

6. Elevated - standpipe

7. Hydropneumatic - steel tank partly below grade

The development of construction costs for all except the last configuration was discussed in Appendix C. Hydropneumatic tanks were treated as steel tanks constructed above grade and sized at four times the required storage. This approach simplified programming requirements and proved to be relatively accurate.

Treated water storage facilities were sized as a function of the total daily treated water demand. A mass diagram of hourly water demands was analyzed to determine storage requirements. In each case, the storage requirement was taken as 0.35 times the total daily demand. This can also be stated as an eight-hour storage requirement. Due to the various configurations considered, the actual tank size should be larger than the storage requirement. The following sizing relationships were used to obtain tank sizes:

For ground level storage and elevated storage by elevated tank

$$\text{CAPT} = 0.35 (\text{DEMC}) \quad (\text{D-1})$$

For elevated storage by standpipe

$$\text{CAPT} = 1.05 (\text{DEMC}) \quad (\text{D-2})$$

For hydropneumatic tanks

$$\text{CAPT} = 1.40 (\text{DEMC}) \quad (\text{D-3})$$

where:

CAPT = tank size in gallons

DEMC = daily treated water demand in gallons

OPERATING AND MAINTENANCE COSTS

Operation and maintenance costs were computed for each element of the system to include the time and material necessary to insure continued reliable operation. System operation at all levels of scale was treated as an incremental addition to a larger system. This assumes that the necessary labor, equipment, and materials would be provided or procured by an existing organization whose overall size and organizational structure would not be significantly changed by the largest system under consideration in this study. Depending on the nature of the final system, the operation responsibility might devolve upon the Columbia Park and Recreation Association, a quasi-governmental body which holds title to the public

space in the Columbia area, including Wilde Lake and its tributaries, or to the Howard County Metropolitan Commission, a public water and sewer authority. In either case, the assumption of a small incremental addition to an existing organization is valid.

Using this approach, labor costs are taken as direct cost plus an allowance for labor fringe benefits. Materials and supplies are costed at procurement cost and equipment at established local rental rates. No provision is made for general or administrative overhead, for billing and collection, or other nondirect items, since it was considered that the addition of the system under study would not appreciably change any of these costs. Wage rates and material costs are estimated on the basis of 1967-1968 prices.

Parametric costs were developed in the same manner as the construction costs, extrapolating a curve from several calculated points. The curves were reduced to tables of points for linear interpolation by the computer program. All operating and maintenance costs are expressed in dollars per day, based on yearly averages of flows and demands.

PRETREATMENT

The principal costs associated with the operation of the pretreatment unit are the replenishment of the various chemicals employed and the periodic removal of accumulated sediment from the sump. Typical chemical requirements for this process are aluminum sulfate (alum) and calcium hypochlorite. Dosage rates have been estimated at 10 mg/L $\text{Al}_2(\text{SO}_4)_3$ and 0.35 mg/L available chlorine. This requires feed rates of 85 pounds of alum and 4.25 pounds of calcium hypochlorite per million gallons of runoff. Other chemicals, such as lime, polyelectrolytes, soda ash, etc., might be required to adequately treat waters in certain locations, but published storm water analyses indicate that the chemicals named will be satisfactory in most applications. All chemicals will be batch-mixed in solution tanks and fed by metering pumps. The feeders will be checked daily and chemicals replenished as required. Computation of operating costs did not include removal of sediment from the pretreatment unit sump, as this operation was combined with cleaning and inspection of the storm water reservoir and described with parametric costs in Appendix C. Chemical costs, electric power costs, and costs associated with regular attention to proper operation are calculated for the pretreatment unit and shown in Tables D-7 and D-8, as well as in Figure D-4. These costs can be identified as operation and maintenance costs for Class "C" treatment and are included in the costs shown for higher levels of treatment.

TABLE D-7. SUMMARY OF OPERATING COST ESTIMATES - TREATMENT

	Capacity Gal/Day	Operation Man/Hrs	Rate	Chemical Cost	\$/Day	Electric Power \$/Day	Misc. Main. Supplies \$/Day	Misc. Oper. Supplies \$/Day	Contingency \$/Day	Total O&M \$/Day
CLASS "C"	100,000	0.65	3.50	2.25	0.45	8.50	0.50	0.20	0.50	12.00
	500,000	2.00	3.50	7.00	2.25	40.50	4.25	2.00	2.50	55.00
	1,000,000	5.00	3.50	17.50	4.50	81.00	35.00	25.00	5.00	168.00
CLASS "B"	100,000	1.50	3.50	5.25	0.75	9.00	1.00	1.00	1.00	18.00
	500,000	3.50	3.50	12.25	3.75	42.50	1.50	1.50	1.50	63.00
	1,000,000	3.50	3.50	12.25	4.25	84.00	1.50	1.50	1.50	105.00
CLASS "A"	100,000	3.00	3.50	10.50	0.85	9.00	1.15	1.50	1.50	25.00
	500,000	10.00	3.50	35.00	4.25	42.50	2.75	3.50	2.00	90.00
	1,000,000	13.00	3.50	45.50	8.50	85.00	3.50	4.50	3.00	150.00
CLASS "AA"	100,000	3.50	3.50	12.25	1.50	10.00	1.50	1.75	3.00	30.00
	500,000	11.00	3.50	38.50	7.00	43.50	5.00	6.00	4.00	105.00
	1,000,000	14.00	3.50	49.00	14.00	86.00	20.00	31.00	5.00	205.00

FINAL TREATMENT

Operation of the final treatment facilities consists of routine inspection of the various items of equipment, daily tests of water quality to verify chemical feed rates and to check automatic monitors, backwashing of filters or strainers, replenishment of chemical feeders, etc. Preventive maintenance is carried out on the various pieces of equipment and minor replacement of components is performed as indicated. Solution-type chemical feeders are used to add a precoat material to the straining operation for Class "B" treatment and to provide a chemical coagulant where required to the Class "A" and Class "AA" plants. It was assumed for costing purposes that this coagulant would be aluminum sulfate (alum), although in practice other materials might be employed and the addition of alkalinity and pH adjusting chemicals, such as lime or soda ash, might be required. Rechlorination will be practiced at all treatment points, using calcium hypochlorite. Feed rates are estimated at 50 pounds $\text{Al}_2(\text{SO}_4)_3$ and 8.5 pounds $\text{Ca}(\text{OCl})_2 \cdot 4\text{H}_2\text{O}$ per million gallons of treated water. Daily attention to all treatment equipment has been assumed.

Estimates also include energy cost for electric power to mixers, metering pumps, sump pumps, etc., as well as power for high service pumping. Table D-7 lists the detailed costs used to develop operating and maintenance costs for various sizes of treatment facilities, and Figure D-4 shows the extrapolated cost curve, as well as the approximated curve. The table of values input to the computer program is shown as Table D-8.

TABLE D-8. OPERATION AND MAINTENANCE
COST VERSUS CAPACITY - TREATMENT

Treatment Capacity (gal/day)	Operating & Maintenance Cost (\$/day)			
	Class "C"	Class "B"	Class "A"	Class "AA"
0.0	0.00	0.00	0.00	0.00
60,000	7.00	9.00	15.00	18.00
100,000	12.00	19.00	24.00	28.00
200,000	24.00	27.00	41.00	48.00
400,000	46.00	52.00	74.00	86.00
600,000	64.00	73.00	103.00	119.00
800,000	80.00	92.00	126.00	146.00
1,000,000	90.00	108.00	145.00	168.00
1,200,000	97.00	119.00	159.00	186.00
1,500,000	106.00	132.00	174.00	206.00

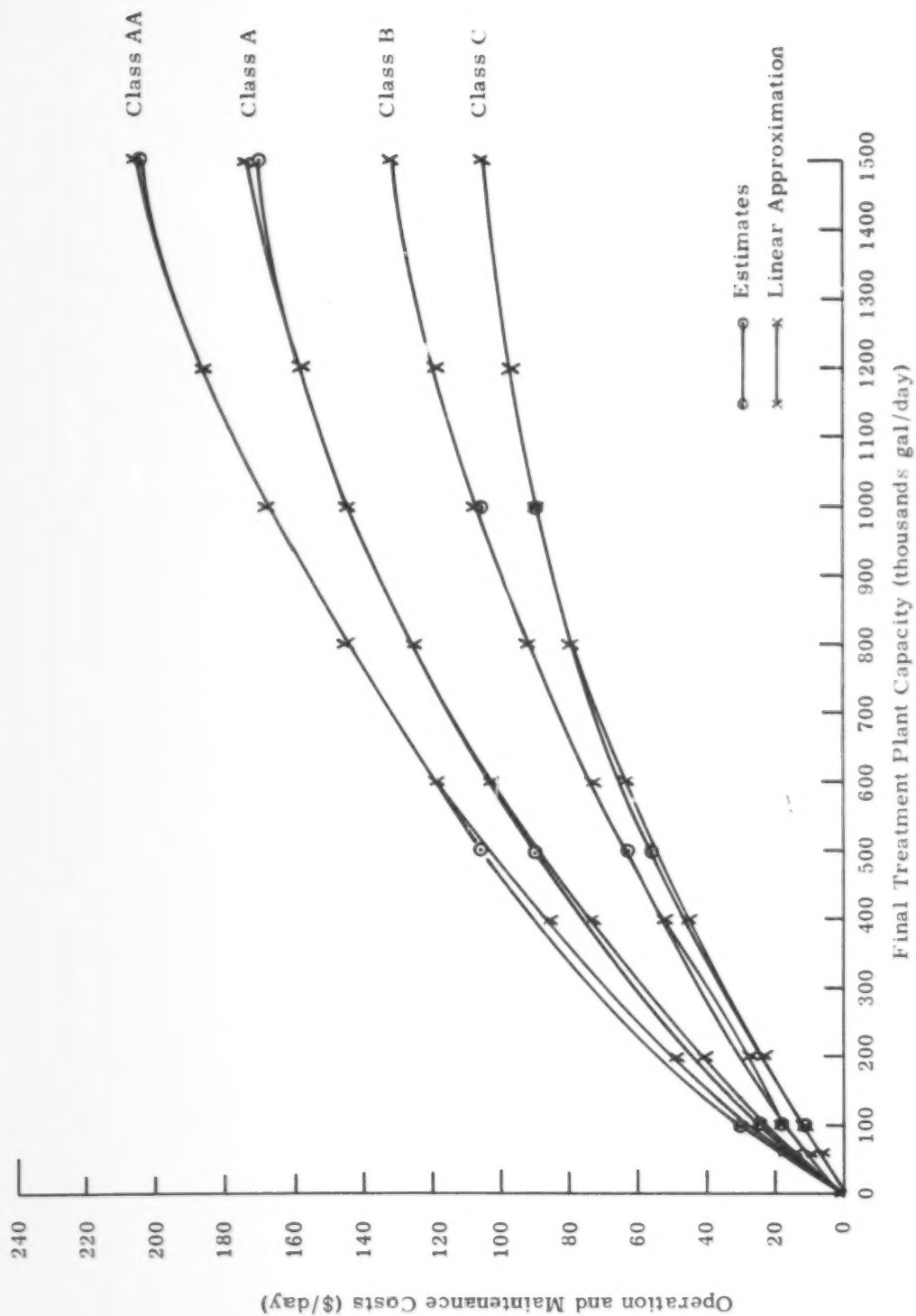


Figure D-4. Operation and Maintenance Cost vs. Capacity - Final Treatment

PUMPING

Operating and maintenance costs of the high service pumping facilities have been included in the Final Treatment costs detailed in Table D-8.

TREATED WATER STORAGE

Operation and maintenance costs were computed separately for each of the seven storage options described previously. These costs consist of periodic painting and repair of the tanks and are a function of the surface area to be maintained. Appendix C describes the methods employed to calculate the surface area for various types of cylindrical tanks. Elevated tanks may be cylindrical, but are more frequently ellipsoidal. In addition, they stand on a pedestal or on legs which have appreciable surface area themselves. A variety of tank shapes and sizes were analyzed.

In calculating total maintenance cost for an elevated tank, a cost of \$0.05 per square foot per year was assumed.